AD-A070 585 GAI CONSULTANTS INC MONROEVILLE PA F/G 13/2
NATIONAL DAM INSPECTION PROGRAM. TWIN LAKES NUMBER 1 DAM (NDS1.--ETC(U)
APR 79 DACW31-79-C-0013

UNCLASSIFIED NL

1 OF 2
AD
A070585

LEVEL

6 National Dam Inspection Program. Twin Lakes Number 1 Dam (NDS-PA-00487, Pennder 65-42), Ohio River Basin, Little Crabtree Creek, Westmoreland County, Pennsylvania. Phase I Inspection Report.

OHIO RIVER BASIN

LITTLE CRABTREE CREEK, WESTMORELAND COUNTY

PENNSYLVANIA

TWIN LAKES NO. 1 DAM

12 109p.

NDS I. D. No. PA-00487

PENNDER I. D. No. 65-42

Contract No. DACW31-79-C-0013

15

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

PREPARED FOR

DEPARTMENT OF THE ARMY

Baltimore District, Corps of Engineers

Baltimore, Maryland 21203

PREPARED BY

GAI CONSULTANTS, INC.

570 BEATTY ROAD

MONROEVILLE, PENNSYLVANIA 15146

11 APR[redacted]79

411 002

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. The spillway design flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition, and the downstream damage potential.

PHASE I REPORT
NATIONAL DAM INSPECTION PROGRAM

ABSTRACT

Twin Lakes No. 1 Dam: NDS I.D. No. PA-00487

Owner: Westmoreland County

State Located: Pennsylvania (PennDER I.D. No. 65-42)

County Located: Westmoreland

Stream: Little Crabtree Creek

Inspection Date: 13 December 1978

Inspection Team: GAI Consultants, Inc.
570 Beatty Road
Monroeville, Pennsylvania 15146

The visual inspection, operational history, and hydrologic/hydraulic analysis indicate the facility is in good condition.

Despite the rehabilitation work performed in 1975, apparent seepage along the downstream toe of the embankment continues, although no measurable flow was observed. The manhole containing the outlet conduit gate valve was found flooded during the inspection. This condition, although not considered a threat to the immediate safety and current operation of the facility, could possibly promote and accelerate corrosion of the valve stem extension particularly at the air-water interface.

Hydrologic and hydraulic calculations indicate the facility will accommodate about 51 percent of the Probable Maximum Flood (PMF) which is considered to be the required spillway design flood (SDF). Consequently, the present spillway is assessed as being inadequate, but not seriously inadequate.

It is recommended that the owner:

a. Immediately develop a plan for emergency operation and a warning system for downstream residents. Included in the plan should be provision for around-the-clock surveillance of the facility during periods of unusually heavy precipitation.

b. Retain the services of a registered professional engineer experienced in hydrology and hydraulics to more accurately assess the spillway systems of both the Upper Donohoe and Twin Lakes No. 1 Dams and their interdependence. Subsequently, implement remedial measures deemed necessary to make the systems hydraulically adequate.

c. Have appropriate agencies evaluate the condition of the highway bridge immediately below the spillway structure and make necessary remedial repairs as failure of the deteriorated bridge during high flows could seriously affect the safe operation of the spillway system.

d. Monitor wet areas across the downstream embankment face on a continual basis. If seepage increases or turbidity occurs, the condition should be evaluated and necessary remedial measures implemented.

e. Develop an operations and maintenance manual for use at the facility. The manual should include a procedure for installing the stop log of the outlet works and provisions for dewatering the gate valve manhole in the event of a valve stem failure and for periodic maintenance.

f. Have the facility inspected on a yearly basis by a registered professional engineer experienced in the design and construction of earth dams to check for hazardous conditions that might develop. The annual inspection should specifically address the seepage condition along the downstream toe of the dam.

GAI Consultants, Inc.

Bernard M. Mihalcin, P.E.

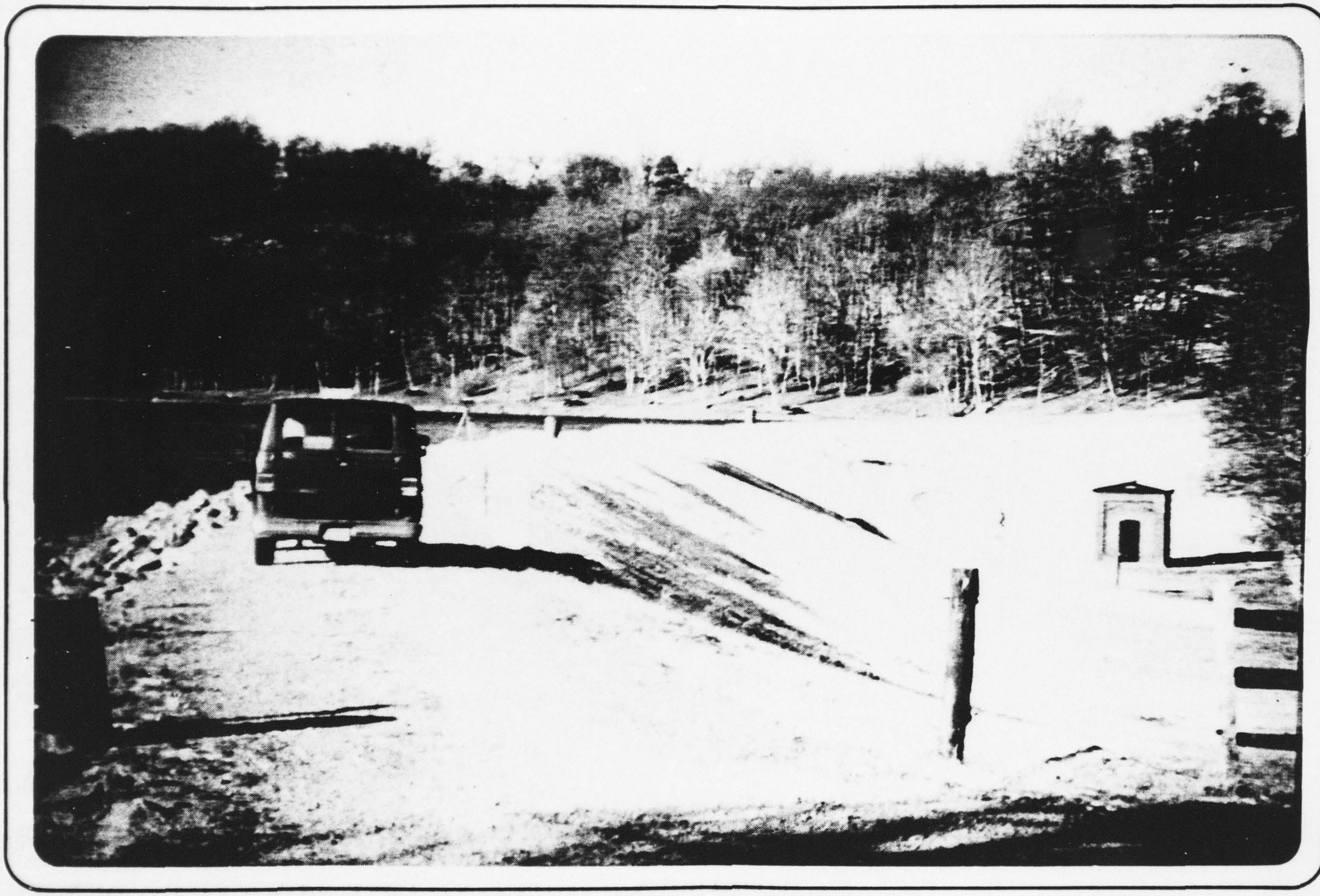
COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
BERNARD M. MIHALCIN
ENGINEER
20371-E

Date 7 MAY 1979

Approved by:

G. K. WITHERS
Colonel, Corps of Engineers
District Engineer

Date 1 Jun 79



OVERVIEW PHOTOGRAPH
V

TABLE OF CONTENTS

TABLE OF CONTENTS

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM
TWIN LAKES NO. 1 DAM
NDI# PA-487, PENNDER# 65-42

SECTION 1
GENERAL INFORMATION

1.0 Authority.

The Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of inspection of dams throughout the United States.

1.1 Purpose.

The purpose is to determine if the dam constitutes a hazard to human life or property.

ABSTRACT

1.2 Description of Project.

a. Dam and Appurtenances. Twin Lakes No. 1 Dam, previously known as the Lower Donohoe Dam, is a recently renovated (1975) earth embankment approximately 950 feet long with a curved crest and a maximum height of about 31 feet. The facility is serviced by an uncontrolled, concrete chute spillway, 80 feet wide at the crest and 25 feet wide at the toe, located at the center of the embankment. The facility is equipped with newly constructed outlet works consisting of a 16-inch diameter ductile iron pipe passing through the embankment to the left of the spillway, a submerged upstream intake structure, control valve, and outlet discharge pipe. The control valve is operated from a manhole accessible from the embankment crest just left of the spillway. An extension on the control valve permits operation without actually descending the manhole.

b. Location. The dam is located in Hempfield and Unity Townships, Westmoreland County, on Little Crabtree Creek, approximately three miles northeast of the city of Greensburg, Pennsylvania. The eastern edge of the village of Luxor lies approximately 2,500 feet downstream of the embankment. The dam, reservoir, and watershed are contained within the Latrobe, Pennsylvania, U.S.G.S. 7.5 minute topographic quadrangle (see Regional Vicinity Map, Appendix G). The coordinates of the dam are N40° 19.6' and W79° 28.5'.

ABSTRACT

c. Size Classification. Small (31 feet high, 470 acre-feet storage capacity at top of dam).

d. Hazard Classification. High (see Section 3.1.e).

e. Ownership. Westmoreland County
Department of Parks and Recreation
P. O. Box 272
Greensburg, Pennsylvania 15601

f. Purpose. Recreation.

g. Historical Data. This historical account of Twin Lakes No. 1 Dam is based on an excellent set of detailed records available from PennDER files, dating from 1911. Twin Lakes No. 1 Dam, formerly known as the Lower Dam or Lower Donohoe Dam, was originally designed and owned by the Jamison Coal and Coke Company and constructed by H. F. Stark of Greensburg, PA in 1908. Later it became the property of Consolidated Coal Company and approximately 15 years ago, ownership of the dam was transferred to Westmoreland County. This dam, in conjunction with the Upper Donohoe Dam (NDI# PA-478) located on the same watershed, was used as the water supply for coal mining and coking operations. The current owners have incorporated both dams into Twin Lakes Park which is used solely for recreational purposes.

State involvement with this facility began in 1911 when a local resident appealed to the Governor for an investigation of the dam. In his letter, the resident called attention to three independent leaks and that the owner planned to increase the reservoir's storage capacity by adding two feet to the spillway crest. The first inspection of record was conducted in 1915 by the Water Supply Commission of Pennsylvania. This inspection revealed that the spillway capacity had been reduced by increasing the spillway crest elevation with the addition of a concrete weir and flashboards. Following the inspection, the Commission ordered the coal company to increase the capacity of the spillway by removing a portion of the spillway crest addition. The owner, however, wished to maintain the increased reservoir storage and proposed to increase the spillway capacity by erecting a concrete wall or parapet along the upstream crest of the embankment which would effectively increase the depth and, therefore, the capacity of the spillway channel. Although the Water Supply Commission was not entirely satisfied, this structure was built in 1916, resulting in compliance with the Commission's earlier order. The parapet, as constructed, consisted of a 7-foot high, 12-inch thick steel reinforced concrete wall embedded in the embankment crest down to the

approximate normal pool level along the length of the embankment. Approximately 2 to 4 feet of earthfill was placed behind the wall to maintain a minimum crest width of 10 feet. The parapet was in no way connected to the concrete core wall.

In 1919, a moderately severe seepage condition on the downstream face of the embankment just right of the spillway was reported. In 1921, the Commission ordered the owner to stop the leakage and restore the dam to safe working order. In 1923 or thereabouts, the owner took positive action to stop the leakage by installing "a concrete wall 16 inches wide and approximately 14 feet deep...along the right spillway abutment." This effort halted the seepage for only a short while. In 1926, seepage and saturated embankment conditions both right and left of the spillway were reported. From about 1926 through 1970, no further effort was made to stop the leakage through the embankment.

Annual inspections for the years 1925 through 1928, 1931 through 1936, 1941 and 1948 report worsening seepage problems and progressive deterioration of concrete surfaces. In 1936, removal of disintegrating concrete at the spillway crest resulted in lowering the spillway crest elevation by 8 inches. The 1948 report indicated the masonry of the parapet was in such a state of disrepair that the effective height of the dam was reduced by about 2 feet on the right abutment and thus, "inviting catastrophe." Conditions at the facility continued to worsen.

In 1964, the general appearance of the facility was "poor." Trees and brush covered the crest and downstream slope. All concrete surfaces were spalling and disintegrating. The condition of the lower toe was described as swampy.

In 1966, following the acquisition of the property by Westmoreland County, a brief inspection was made with a delegation from the Westmoreland County Recreation Commission. As a result of this inspection, state engineers urged rehabilitation of the facility at the earliest possible date.

Early in 1970, the Pittsburgh District of the U. S. Army Corps of Engineers inspected the facility. Their report reiterated, in detail, all the problem areas previously described. The Corps' report stated that "the dam was in rather poor condition due to the seepage, the lack of repair to the wall (parapet) and spillway and the questionable condition of the conduit controls but, that it appeared that no emergency work was necessary." The County Commission was

advised to retain the services of a private consultant in order to obtain recommendations as to required remedial action.

In the summer of 1970 following the Corps' inspection of the dam, state engineers again inspected the facility. Their report indicated that nothing had been done relative to their 1966 recommendation for early rehabilitation. As a consequence, the 1970 inspection report by the state engineer declared the structure to be unsafe and recommended immediate action to correct the situation. The report also suggests "the lake (should) be lowered to a safe point until action is taken to correct the deficiencies." Quickly following this inspection report, the Water and Power Resources Board ordered the Westmoreland County Recreation Commission to drain the lake and submit a plan for rehabilitation. If the County Commission decided not to make repairs, the embankment would be breached sufficiently that it would offer no impedence to the flow of the stream. The Westmoreland County Recreation Commission responded by draining the lake in the fall of 1970. Shortly thereafter, the Commission retained Geo-Mechanics, Inc., consulting engineers of Belle Vernon, Pennsylvania to investigate the dam and appurtenances and to develop a plan for rehabilitating the facility.

Rehabilitation work was initiated on May 5, 1975, and completed on July 31, 1975. Extensive modifications of the facility were made during this reconstruction. Compacted fill was added to the upstream slope of the embankment in order to reduce the slope to 3 horizontal to 1 vertical and to provide a longer seepage path. A vertical granular drain was constructed along the downstream slope of the embankment to control seepage and lower the phreatic surface, thus increasing the stability of the downstream slope. The original spillway was demolished and completely rebuilt. Spillway capacity was increased by constructing an 80-foot wide concrete chute spillway with increased freeboard and improved flow characteristics over the original design. A completely new outlet works was installed. This consisted of an intake structure, a 16-inch diameter ductile steel outlet conduit, valve pit and access manhole on the crest of the embankment and a downstream outlet discharging into the lower end of the spillway channel. The old intake structure was demolished and all the original conduits passing through the embankment were plugged. An 18-inch layer of riprap placed on a 6-inch thick gravel and sand cushion was provided on the upstream embankment slope 3 feet above and below the normal pool elevation.

Following rehabilitation work, the outlet conduit was closed in August 1975 and by the end of October 1975, the

reservoir had filled to within 6 inches of normal pool. Since reconstruction, the dam has been well maintained and has functioned adequately.

1.3 Pertinent Data.

a. Drainage Area (square miles). 1.5 (local)
1.9 (total)

b. Discharge at Dam Site. Discharge records are not available. The Jamison Coal and Coke Company, however, reported that during the spring flood in 1936 the maximum depth of water in the spillway was 12 inches occurring on March 17, 1936. At this time, both a 4-inch and 14-inch diameter blowoff conduits were full open.

Outlet Conduit at Operating Pool Elevation - Discharge curve not available.

Emergency Spillway Capacity at Top of Dam Pool ≃ 2000 cfs.

c. Elevation (feet above mean sea level). The following elevations were obtained through field measurements based on the elevation of the service spillway at 1094 feet as reported in PennDER files. Elevations marked on Figures 3 through 7 are low by 3 feet. Elevation of normal pool shown on Figure 2 is incorrect and should read 1091 feet.

Top of Dam	1097.7
Maximum Design Pool	Not known
Maximum Pool of Record	Not known
Normal Pool	1094
Service Spillway Crest	1094
Outlet Upstream Invert	1075
Outlet Downstream Invert	1069
Streambed at Dam Centerline	1065
Maximum Tailwater	Not known

d. Reservoir Length (miles).

Top of Dam	0.35
Normal Pool	0.35

e. Storage (acre-feet).

Normal Pool	340
Top of Dam	470

f. Reservoir Surface (acres).

Normal Pool	33
Top of Dam	39

g. Dam.

Type	Earthen embankment built with upstream riprap slope protection and a concrete core wall.
Length	950 feet (field measured)
Height	31 feet (field measured)
Downstream Slope	2H:1V (crest to toe, field measured)
Upstream Slope	3H:1V (exposed free-board zone, field measured)
Zoning	None. Internal drainage added during rehabilitation in 1975.
Impervious Core	One-foot thick, steel reinforced, concrete core wall approximately 850 feet in length is reportedly located beneath the downstream crest of the embankment. The core wall has a reported maximum height of 32 feet and extends 2 feet below the original ground surface. The wall was not disturbed during the 1975 rehabilitation work (see Figure 5).
Cutoff	The 1915 inspection report indicates that there is a clay puddle

		trench under the upstream toe of the original embankment which is 10 feet wide, about 18 feet high, and extends below the existing ground surface about 15 feet.
	Grout Curtain	None indicated.
h.	<u>Diversion and Regulating Tunnels</u>.	None.
i.	<u>Spillway</u>.	
	Type	The original service spillway was completely replaced during the 1975 rehabilitation. The new spillway is an uncontrolled, concrete chute with no weir or provision for boarding. The spillway is 80 feet wide along the crest narrowing to 25 feet at the toe.
	Crest Elevation	1094
	Crest Length	80 feet (field measured).
j.	<u>Outlet Conduit</u>.	
	Original Supply and Blowoff Pipes	The original 12-inch diameter supply line and 14-inch diameter blowoff pipe were sealed with concrete as part of the 1975 rehabilitation work.
	New Blowoff Pipe	A 16-inch diameter, ductile steel pipe, encased by 8 inches of reinforced concrete for the full length of

	≃ 197 feet (inlet invert to outlet invert), was installed in 1975 (see Figure 7).
Closure	Drawdown control is provided by a 16-inch diameter gate valve located at the base of the access manhole.
Regulating Facilities	The gate valve access manhole is located on the crest of the embankment, just to the left of the spillway. The 48-inch diameter reinforced concrete pipe manhole contains one 16-inch diameter gate valve control at the base. The gate valve control is provided with an extension to permit operation from the crest of the dam without descending the manhole (see Figure 7).

SECTION 2
ENGINEERING DATA

2.1 Design.

a. Design Data Availability and Sources. No design reports or calculations pertaining to the original facility are available. One drawing is available, dated August 2, 1916, that shows a typical embankment section and the proposed parapet addition. The drawing is contained in PennDER files. Detailed geotechnical and hydraulic studies were performed by Geo-Mechanics, Inc., of Belle Vernon, Pennsylvania, for the extensive rehabilitation work performed in 1975. The studies are summarized in their report entitled, "Rehabilitation Investigation, Lower Dam, Twin Lakes Park, Greensburg, Pennsylvania," dated December 10, 1973. The above available data were reviewed by the inspection team.

b. Design Features.

1. Embankment. According to information supplied to the Water Supply Commission of Pennsylvania in 1915, the original embankment was constructed of rolled earth placed in layers on an earth foundation prepared by removing the surface soil. A reinforced concrete core wall with a maximum height of 32 feet, a length of 850 feet, top width of 1 foot, and a base width of 1 foot was constructed at the position of the downstream crest of the dam to control rodent burrows. The original embankment was constructed with a downstream slope of 1.5H:1V and an upstream slope of 2H:1V. Upstream slope protection was provided with hand-placed riprap paving. A clay puddle, which is 10 feet wide, about 18 feet high, and extends below the original ground surface about 15 feet, trench was placed along the upstream toe of the embankment. The crest width of the original structure was 10 feet.

In 1975, major modifications of the embankment were made in accordance with the rehabilitation design prepared by Geo-Mechanics, Inc. Substantial earthfill was added to the upstream slope in order to flatten the slope angle to 3H:1V. An 18-inch layer of riprap was placed on the upstream slope on a 6-inch gravel and sand cushion for a vertical distance of 3 feet above and below normal pool elevation. On the downstream slope, a 4-foot wide vertical drain with collection pipe was constructed to help lower the phreatic surface in the embankment and to control seepage. The toe area below the drain was rebuilt with compacted earthfill. The toe seepage drain discharges into the right side of the spillway channel directly opposite the outlet

conduit discharge. The downstream slope was regraded to reduce the slope angle to 2H:1V. A plan of the embankment showing the overall configuration of the rehabilitated facility is presented in Figure 3, Appendix F.

2. Appurtenant Structures.

a) Spillway. The rehabilitated spillway is an uncontrolled, concrete chute. The new spillway was built to approximately the same shape as the original, but with a lower crest elevation and modified shape. The spillway is 80 feet wide at the crest and narrows to 25 feet at the toe (see Figure 7 and Photograph 5).

b) Outlet Works. The rehabilitated outlet works (blowoff) consists of one 16-inch diameter ductile iron pipe encased in 8 inches of reinforced concrete carried through the dam. The 16-inch diameter outlet conduit is approximately 197 feet in length and passes through the embankment just left of the spillway. Flow through the conduit is controlled by a 16-inch diameter gate valve situated at the base of the access manhole located along the crest of the embankment. The submerged inlet is also equipped with a trash rack and stop log slot (see Figure 7).

Discharge from the outlet channel is directed into the base of the spillway channel (see Photograph 4).

c. Design Data and Procedures. No design data are available for the original facility. Rehabilitation design procedures and parameters are summarized in the consultants' rehabilitation report.

1. Hydrology and Hydraulics. Spillway capacity calculations for the old structure and a proposed renovated structure are contained in an appendix of the consultants' rehabilitation report.

Correspondence in PennDER files indicates subsequent modifications were recommended by PennDER which presumably provided for a spillway sized to pass a peak flow as determined by the Pennsylvania "C" Curve criteria.

2. Embankment. In 1975, test borings were drilled and bag samples secured for the purpose of developing rehabilitation design parameters. Undisturbed Shelby tube samples were secured from six of the embankment borings. Additional bag samples were obtained from the borrow areas. Standard penetration resistance tests were performed on all borings. In addition, water pressure testing was performed in six of the embankment borings. Direct shear and permeability tests were run on the Shelby tube samples whereas, laboratory compaction, constant head permeability,

mechanical analysis, and hydrometer tests were performed on the bag samples.

Following laboratory testing, subsurface cross-sections and profiles were developed for use in the seepage, settlement, and stability analyses. The seepage analysis was performed via flow net method. The maximum settlement of new fill materials was estimated not to exceed one percent. For the stability analysis, the Swedish Circular Arc Method was used to analyze the stability of both the existing and rehabilitated embankment.

a) Seepage Analysis. (Edited excerpt from Geo-Mechanics' Report). A soil permeability value of 1.3 x 10^{-6} cm/sec for all soils in the embankment and foundation was selected based on the laboratory soil tests. This value was for vertical permeability. The horizontal permeability was assumed to be 9 times the vertical permeability due to horizontal stratification and anisotropy in the soil mass. Water pressure test data indicate the permeability of the underlying bedrock varied from 1.5 x 10^{-3} cm/sec to almost 0. The lowest permeability values were associated with the rock strata nearest the top of rock. Therefore, the top of bedrock was considered to be impervious for the seepage analysis of the embankment. Furthermore, significant loss of water through the more permeable bedrock strata is considered unlikely.

Using the above data and assumptions, a flow net analysis was performed to estimate seepage from the impoundment. From the analysis, a total daily flow through the embankment of about 785 gallons was calculated. This amount of seepage will not significantly affect the storage of water in the reservoir and is considered acceptable.

b) Settlement Analysis. (Edited excerpt from Geo-Mechanics' Report). Significant settlement of the proposed embankment is not anticipated. Backfilling areas of over-excavation plus new embankment fill is not expected to raise the total height of fill at any point more than 15 feet. Settlement in this fill is estimated not to exceed one percent or about 2 inches. Additional settlement of the embankment due to consolidation of the foundation material is expected to be small because of over-excavation and stabilization of soft areas.

c) Stability Analysis. (Edited excerpt from Geo-Mechanics' Report). To determine the stability of the rehabilitation design, the Swedish Circular Arc Method was used to analyze the stability of both the original and rehabilitated embankment. For the original embankment, calculations of stability over the long term result in a

factor of safety of approximately 1.0. After reconstruction, the long-term stability of the downstream slope will be increased to an acceptable factor of safety of approximately 1.75. The increase in stability is due primarily to the vertical drain which lowers the phreatic surface in the dam. In both of the above analyses, the effect of the existing cutoff wall was neglected because over the long term, the wall will further deteriorate to a soil-like mass.

Modified Proctor curves, grain-size distribution curves, boring logs, seepage, stability analysis, and drawings appear in the appendix of the consultant's rehabilitation report.

3. Appurtenant Structures.

a) Spillway. Based on construction drawings and data available from the owner and PennDER, the spillway appears to be adequately designed and constructed. No design calculations were made available to the inspection team for review.

b) Outlet Works. Review of construction drawings indicates that the outlet works was designed in accordance to generally accepted engineering practice and contains provisions for blocking flow at the upstream inlet, if required. No calculations for sizing the outlet conduit or drawdown curves were available for review.

2.2 Construction.

No construction records are available for the original embankment. Construction data, however, are available for the rehabilitation work performed in 1975. Related to this work are construction drawings, construction specifications and progress reports (with related correspondence). These data are available for review from the consultants' files, PennDER's files, and/or from the files of the Westmoreland County, Department of Parks and Recreation. Construction was field monitored on a full-time basis by the consultant.

2.3 Operating Records.

No pool level, rainfall, or discharge records are kept for this facility. Some records, however, are available from PennDER's files detailing performance of the original facility during periods of prolonged rainfall; i.e., the 5.9-inch rainfall in March of 1936.

2.4 Other Investigations.

No engineering related investigations subsequent to the rehabilitation work have been conducted other than regular inspections of the facility by PennDER personnel.

2.5 Evaluation.

Sufficient data are available to make a Phase I assessment of the facility. A comprehensive rehabilitation investigation was conducted in 1973. A summary report, construction drawings, specifications, and construction progress reports are available for review.

SECTION 3
VISUAL INSPECTION

3.1 Observations.

a. General. The general appearance of this facility suggests the dam and its appurtenances are currently in good condition.

b. Embankment. The visual inspection suggests the embankment to be in good condition. Minor seepage, however, was observed along the downstream toe, particularly to the right of the spillway section (see Photograph 3). Despite the localized saturated areas, no signs of sloughing, erosion, or free flowing water were observed on or immediately below the embankment. A slag-gravel roadbed protects the crest against damage from occasional vehicular use. The downstream slope is covered with grass that requires little maintenance other than occasional mowing (see Photograph 1). The riprap is durable and well graded and provides adequate slope protection.

c. Appurtenant Structures.

1. Spillway. Based on visual observations, the spillway is in good condition (see Photograph 5). No concrete deterioration was evident.

2. Outlet Works. Complete submergence of the inlet to the outlet conduit precluded the possibility of visual inspection.

A manhole located on the crest just left of the spillway provides access to a 30-foot vertical reinforced concrete shaft, the bottom of which houses the manually operated gate valve. An extension on the valve stem permits operation from the top of the manhole. At the time of the inspection, the manhole was filled with water to elevation 1085.1 or approximately 5.9 feet below normal pool (see Photograph 10). Despite the flooded condition, the valve was operated by county personnel in the presence of the inspection team. The valve appeared to function normally as water was observed issuing from the outlet conduit at the base of the spillway channel (see Photograph 4).

d. Reservoir Area. The general area surrounding the reservoir is characterized by gentle to moderate slopes that are partially wooded. No signs of slope distress were observed (see Photograph 9).

e. Downstream Channel. Immediately below the service spillway chute and the outlet conduit is a small concrete bridge for a secondary road that crosses the channel. The bridge over the channel is in a severe state of disrepair displaying badly deteriorated concrete surfaces, displaced guard rails, and disintegration of the slab underpinnings. A collapse of the bridge under high flows could create a major downstream channel obstruction and cause serious backwater problems.

Flow discharged into the stream beyond the embankment follows a gently sloping course through a lightly wooded rural area. Approximately 2,500 feet downstream of the dam is the first house that could be affected by a dam failure (see Photograph 11). Many mobile homes are also located within the floodplain in this area. It is estimated that within this reach more than one hundred people could be affected by an embankment breach. Therefore, the hazard classification of the facility is considered to be "high".

Little Crabtree Creek merges with Crabtree Creek approximately 2.7 miles downstream of the embankment. Approximately 3.6 miles downstream of the dam, Crabtree Creek flows within the flood pool boundary of the Loyalhanna Reservoir, a major flood control project.

3.2 Evaluation.

Observations made during the visual inspection suggest that the overall condition of the facility is good. The only deficiencies noted were minor seepage along the downstream toe (especially right of the spillway), the flooded access manhole, and the potential downstream obstructions in the spillway channel immediately below the dam.

SECTION 4
OPERATIONAL PROCEDURES

4.1 Normal Operational Procedure.

According to the owner's representative, there are no formal operational procedures at the facility and the facility is essentially self-regulating. Under the present procedure, the outlet conduit is opened only when there is need to draw down the reservoir.

4.2 Maintenance of Dam.

Required routine maintenance is performed by Westmoreland County personnel on an unscheduled basis. The general appearance of the facility indicates no specific areas of neglect.

4.3 Maintenance of Operating Facilities.

There is no formal maintenance program for the operating facilities. The main valve on the outlet conduit is presently inaccessible due to the flooded condition of the access manhole. An extended stem, however, permits opening of the valve from the top of the manhole.

4.4 Warning Systems.

There are no formal warning systems in effect.

4.5 Evaluation.

The facility is designed to be self-regulating and requires minimal maintenance. Formal procedures are recommended, however, to ensure adequate maintenance and continued operability of the operating facilities. No formal warning system is in effect.

SECTION 5
HYDROLOGIC/HYDRAULIC EVALUATION

5.1 Design Data.

A hydrologic/hydraulic analysis of both the proposed and existing spillways (before reconstruction) was performed by Geo-Mechanics, Inc. Pertinent data are available in their report entitled, "Rehabilitation Investigation, Lower Dam, Twin Lakes Park, Greensburg, Pennsylvania." The final design of the spillway was made in accordance with PennDER directives presumably to pass the peak flow as determined from Pennsylvania "C" Curve criteria.

5.2 Experience Data.

Discharge records are not available for the existing facility.

5.3 Visual Observations.

On the date of inspection, no conditions were observed that would indicate the spillway and outlet system would not perform satisfactorily during a flood event.

5.4 Method of Analysis.

The facility has been analyzed in accordance with the procedures and guidelines established by the U. S. Army Corps of Engineers, Baltimore District, for Phase I hydrologic and hydraulic evaluations. The analysis has been performed utilizing a modified version of the HEC-1 computer program developed by the U. S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, California. Analytical capabilities of the program are briefly outlined in the preface contained in Appendix C.

5.5 Summary of Analysis.

a. Spillway Design Flood (SDF). In accordance with procedures and guidelines contained in the National Guidelines for Safety Inspection of Dams for Phase I investigations, the Spillway Design Flood (SDF) for Twin Lakes No. 1 Dam ranges between the 1/2 PMF (Probable Maximum Flood) and the PMF. This classification is based on the relative size of the dam (small), and the potential hazard of dam failure

to downstream residents (high). Due to the presence of impoundment of questionable integrity just upstream, and the high damage potential of dam failure to downstream residents, the SDF for this facility is considered to be the PMF.

b. Results of Analysis. The Twin Lakes No. 1 Dam was evaluated under assumed normal operating conditions. That is, the reservoir was initially at its normal pool or spillway elevation of approximately 1094.0 feet, with the low level blowoff conduit closed. The spillway is a concrete chute channel with a flat critical flow control crest.

The Twin Lakes No. 1 Reservoir has four major independent sources of inflow. One source is the outflows from the Upper Donohoe Dam located just upstream from the reservoir, and the other three sources are three streams which enter the reservoir at distinctly different points. Since the three sub-basins which are drained by the three streams are very similar in area and other physical characteristics, the local reservoir inflow unit hydrograph was based on the features of one representative sub-basin (Appendix C, Sheet 2).

In addition, since Upper Donohoe Dam provides one of the sources of reservoir inflow, it was also evaluated in this study. The Upper Donohoe Dam was analyzed such that its reservoir was initially at its normal pool or spillway elevation of approximately 1126.0 feet, with the low level blowoff conduit closed. The spillway is presently an unlined chute channel with a flat concrete critical flow control crest. A large railroad embankment with a small culvert for flow passage is located just upstream from the Upper Donohoe Reservoir. In order to account for the effects of this embankment on the Upper Donohoe Reservoir inflows and, thus, outflows, the embankment was considered to function like a dam in the analysis, with the small culvert providing the only means of discharge.

All pertinent engineering calculations relative to the evaluations of both Twin Lakes No. 1 Dam and Upper Donohoe Dam are provided in Appendices C and C-1, respectively.

Overtopping analysis (using the Modified HEC-1 computer program) indicated that the discharge/storage capacity of Twin Lakes No. 1 Dam could accommodate only about 51 percent of the PMF prior to overtopping of the dam (Appendix C, Summary Input/Output Sheets, Sheet L). The peak PMF (SDF) inflow into Twin Lakes No. 1 Reservoir of about 3930 cfs was virtually unaffected by the discharge/storage capabilities of the dam and reservoir since the resulting peak outflow was about 3910 cfs (Summary Input/Output Sheets, Sheets I and J). Under the PMF, the Twin Lakes No. 1 Dam embankment

was overtopped for approximately 5.0 hours, with a maximum depth of inundation of about 0.9 feet (Summary Input/Output Sheets, Sheet L). It should be noted that if the embankment crest was level at the design elevation of 1098.0 feet, the discharge/storage capacity of the facility could accommodate about 55 percent of the PMF (Appendix C, Sheet 7, Note 6; and Summary Input/Output Sheets, Sheet L).

The Upper Donohoe Dam controls the Twin Lakes No. 1 Reservoir inflows from about 20 percent of its total basin area. The peak PMF outflow from Upper Donohoe Dam was about 640 cfs, the peak 1/2 PMF outflow was about 250 cfs (Summary Input/Output Sheets, Sheet G). Had the Upper Donohoe Dam not been present, the peak PMF inflow from its drainage basin into the Twin Lakes No. 1 Reservoir would have been at least 750 cfs, and the peak 1/2 PMF inflow would have been at least 410 cfs (Summary Input/Output Sheets, Sheet E). Therefore, the analysis indicates that if the storage potential of the Upper Donohoe Reservoir was removed and all other aspects of the total basin remained the same, the discharge/storage capacity of the Twin Lakes No. 1 Dam would probably accommodate less than 50 percent of the PMF. Also, if either the capacity of the railroad embankment culvert located just upstream from Upper Donohoe Reservoir or the spillway capacity of Upper Donohoe Dam was significantly increased, the discharge/storage capacity of the Twin Lakes No. 1 Dam would possibly accommodate less than 50 percent of the PMF. Thus, the ability of Twin Lakes No. 1 Dam to pass and/or store a flood of about 1/2 PMF magnitude or greater is dependent on the ability of Upper Donohoe Dam to handle the same frequency flood. To emphasize this dependency further, the failure of the Upper Donohoe Dam (which can accommodate about 54 percent of the PMF; Summary Input/Output Sheets, Sheet L) will most likely result in the failure of Twin Lakes No. 1 Dam.

5.6 Spillway Adequacy.

Hydrologic and hydraulic analyses indicate that under existing normal operating conditions, the spillway system of the Twin Lakes No. 1 Dam can accommodate approximately 51 percent of the PMF. Since the SDF for the facility is the full PMF, the spillway system is considered inadequate, but not seriously inadequate. Furthermore, the adequacy (or inadequacy) of the system is highly dependent on the existence and hydraulic characteristics of the Upper Donohoe Dam and upstream railroad embankment.

SECTION 6
EVALUATION OF STRUCTURAL INTEGRITY

6.1 Visual Observations.

a. Embankment. Based on visual observations, the rehabilitated embankment appeared to be in good condition. Despite evidence of minor seepage and localized areas of saturation along the downstream toe of the embankment, no sloughing or erosion of embankment materials was observed. The embankment crest (road), the upstream riprap slope, and the grass covered downstream slope are designed for minimal maintenance.

b. Appurtenant Structures.

1. Spillway. The rehabilitated spillway appeared to be in good condition. No major cracks or spalling of concrete surfaces were in evidence.

2. Outlet Works. The inlet conduit, trash rack, and stop log assembly could not be observed as these structures are submerged. The outlet end of the discharge conduit was observed to be in good condition. During the inspection, the 16-inch diameter gate valve was operated and shown to function satisfactorily. The only deficiency noted was the flooded condition of the access manhole. Entering the manhole to work on the valve or replace the valve stem extension would require pumping out the manhole shaft.

6.2 Design and Construction Techniques.

Available engineering data indicate the rehabilitated facility has been adequately designed and constructed in accordance with modern acceptable engineering practices.

6.3 Past Performance.

According to available records, this facility has performed satisfactorily during its first 70 years despite a long history of spillway inadequacy, severe seepage through the embankment, and deteriorating concrete structures. A complete rehabilitation of the facility was made in 1975. Rehabilitation resulted in an increased spillway capacity, reduced seepage, and improved stability of the embankment. According to Westmoreland County officials, the facility has functioned satisfactorily following rehabilitation.

6.4 Seismic Stability.

The dam is located in Seismic Zone No. 1, and is, thus, subject to minor earthquake induced forces. It is thought that the static stability of the structure is sufficient to withstand such forces; however, no calculations or investigations were performed to confirm this opinion.

SECTION 7
ASSESSMENT AND RECOMMENDATIONS FOR REMEDIAL MEASURES

7.1 Dam Assessment.

a. Safety. The visual inspection, operational history, and available engineering data suggest that the facility is in good condition.

Hydrologic and hydraulic calculations indicate the facility will accommodate approximately 51 percent of the PMF assuming normal operating conditions at both the Twin Lakes No. 1 and Upper Donohoe Reservoirs. Consequently, the dam would be overtopped if subjected to the inflow resulting from a PMF event. As the facility's hazard rating is "high" and the SDF is considered to be the full PMF, the present spillway is assessed as being inadequate, but not seriously inadequate. The assessment is also highly dependent on the existence and hydraulic characteristics of the Upper Donohoe Dam and upstream railroad embankment.

Despite the rehabilitation work performed in 1975, seepage along the downstream toe of the embankment continues. Although no measurable flow was observed at the time of the inspection, some isolated areas, particularly to the right of the spillway, were saturated. The condition is currently of minor concern but should be addressed in future inspections.

Flooding of the gate valve control manhole is undesirable; however, it does not affect the current operation and safety of the facility. Prolonged submergence of the gate valve is likely to accelerate corrosion of the valve stem extension particularly at the air-water interface. This could lead to failure of the valve stem when attempting to operate the valve.

The deteriorated condition of the highway bridge immediately below the spillway structure was noted. Its condition should be evaluated and remedial measures taken as possible failure of the bridge during high flows could seriously affect the safe operation of the spillway system.

b. Adequacy of Information. The available data are considered sufficient to make an accurate Phase I assessment of the facility.

c. Urgency. An emergency plan and warning system should be implemented immediately. Other recommendations and remedial measures listed below should be implemented as soon as possible.

d. Necessity for Additional Investigation. It is recommended that the owner retain the services of a professional engineer experienced in hydrology and hydraulics to more accurately access the spillway system of both the Upper Donohoe and Twin Lakes No. 1 Dams and their interdependence.

7.2 Recommendations/Remedial Measures.

It is recommended that the owner:

a. Immediately develop a plan for emergency operation and a warning system for downstream residents. Included in the plan should be provision for around-the-clock surveillance of the facility during periods of unusually heavy precipitation.

b. Retain the services of a professional engineer experienced in hydrology and hydraulics to more accurately assess the spillway systems of both the Upper Donohoe and Twin Lakes No. 1 Dams and their interdependence. Subsequently, implement remedial measures deemed necessary to make the systems hydraulically adequate.

c. Have appropriate agencies evaluate the condition of the highway bridge immediately below the spillway structure and make necessary remedial repairs as failure of the deteriorated bridge during high flows could seriously affect the safe operation of the spillway system.

d. Monitor wet areas across the downstream embankment face on a continual basis. If seepage increases or turbidity occurs, the condition should be evaluated and necessary remedial measures implemented.

e. Develop an operations and maintenance manual for use at the facility. The manual should include a procedure for installing the stop log of the outlet works and provisions for dewatering the gate valve manhole in the event of a valve stem failure and for periodic maintenance.

f. Have the facility inspected on a yearly basis by a registered professional engineer experienced in the design and construction of earth dams to check for hazardous conditions that might develop. The annual inspection should specifically address the seepage condition along the downstream toe of the dam.

APPENDIX A

CHECK LIST - ENGINEERING DATA

CHECK LIST
ENGINEERING DATA
PHASE I

NAME OF DAM: Twin Lakes No. 1 Dam

NDI#: PA-487 PENNDER#: 65-42

ITEM	REMARKS NDI# PA - 487
PERSONS INTERVIEWED AND TITLE	Adrian Horvath - Maintenance Development Coordinator William Paxton - Planning Coordinator (Landscape Architect) Westmoreland County, Department of Parks and Recreation
REGIONAL VICINITY MAP	See Appendix G. U.S.G.S. 7.5 minute series quadrangle, Latrobe, Pennsylvania, dated 1954 and photorevised in 1969.
CONSTRUCTION HISTORY	Construction history of the original embankment is inferred from PennDER correspondence (see Section 1.2.g). Rehabilitation Design: Geo-Mechanics, Inc. Construction: Weaver Coal and Construction Company, started 5 May 1975; closed conduit August 1975. Inspection: Geo-Mechanics, Inc.
AVAILABLE DRAWINGS	See Appendix F, Figures 2 through 7. Several additional drawings relative to the original facility are contained in PennDER files.
TYPICAL DAM SECTIONS	See Appendix F, Figures 3, 4, and 5.
OUTLETS: PLAN DETAILS DISCHARGE RATINGS	See Appendix F, Figure 7. Discharge rating curves not available.

ITEM	REMARKS NDI# PA - 487
SPILLWAY: PLAN SECTION DETAILS	See Appendix F, Figures 3 and 6.
OPERATING EQUIPMENT PLANS AND DETAILS	See Appendix F, Figure 7.
DESIGN REPORTS	No design reports available pertaining to the original embankment. Rehabilitation design report entitled, "Rehabilitation Investigation, Lower Dam, Twin Lakes Park, Greensburg, Pennsylvania," by Geo-Mechanics, Inc., of Belle Vernon, Pennsylvania, available from Westmoreland County, Department of Parks and Recreation, Greensburg, Pennsylvania.
GEOLOGY REPORTS	Contained within the above-mentioned rehabilitation design report.
DESIGN COMPUTATIONS: HYDROLOGY AND HYDRAULICS STABILITY ANALYSES SEEPAGE ANALYSES	Contained within the above-mentioned report. Revision correspondence available from Westmoreland County, Department of Parks and Recreation.
MATERIAL INVESTIGATIONS: BORING RECORDS LABORATORY TESTING FIELD TESTING	Contained within the rehabilitation design report and also displayed on the drawings, see Appendix F, Figures 3, 4, and 5. Field density test data and concrete cylinder test data available from the owner.

ITEM	REMARKS NDI# PA - 487
BORROW SOURCES	Borrow materials were secured from residual soils within the impoundment area.
POST CONSTRUCTION DAM SURVEYS	Final inspection was performed in 1976. Minor modifications were suggested; one with respect to irregularities on the downstream slope near the left abutment and the other with respect to a deteriorating concrete patch on the spillway crest.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	None since reconstruction.
HIGH POOL RECORDS	Elevation of highest pool since reconstruction is not known.
MONITORING SYSTEMS	None.
MODIFICATIONS	Investigated by Geo-Mechanics, Inc., in 1973. Complete rehabilitation work included placing additional embankment fill on the upstream and downstream slopes, installing a new spillway and outlet works, constructing a vertical drain on the downstream slope, and placing riprap on the upstream slope.

ENGINEERING DATA (CONTINUED)

ITEM	REMARKS NDI# PA - 487
PRIOR ACCIDENTS OR FAILURES	None.
MAINTENANCE: RECORDS MANUAL	There are no formal maintenance or operation programs in effect at this facility. Informal maintenance is accomplished through periodic mowing, visual inspection, etc., by county park personnel.
OPERATION: RECORDS MANUAL	Pool elevation, daily discharge, or operational records are not kept for this facility. No formal operational manual is available.
OPERATIONAL PROCEDURES	There are no formal operational procedures associated with this facility. Excess inflow is discharged through the self-regulated emergency spillway. The outlet works are operated manually for drawdown purposes only.
WARNING SYSTEM AND/OR COMMUNICATION FACILITIES	Informal contacts with the local fire department are maintained by park personnel. The park police are also aware of the potential hazard to the downstream population. There is no job assignment for someone to watch the reservoir during periods of high rainfall.
MISCELLANEOUS	

CHECK LIST
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

NDI ID # PA-487

PENN DER ID # 65-42

SIZE OF DRAINAGE AREA: 1.5 square miles (local); 1.9 square miles (total)

ELEVATION TOP NORMAL POOL: 1094 STORAGE CAPACITY: 340 acre-feet

ELEVATION TOP FLOOD CONTROL POOL: - STORAGE CAPACITY: -

ELEVATION MAXIMUM DESIGN POOL: - STORAGE CAPACITY: -

ELEVATION TOP DAM: 1097.7 STORAGE CAPACITY: 470 acre-feet

SPILLWAY DATA

CREST ELEVATION: 1094

TYPE: Uncontrolled concrete rectangular channel

WIDTH: 80 feet

LENGTH: 131 feet

SPILLOVER LOCATION: embankment center

NUMBER AND TYPE OF GATES: None

OUTLET WORKS

TYPE: 16-inch diameter ductile steel conduit encased by 8 inches of reinforced concrete

LOCATION: left of the emergency spillway

ENTRANCE INVERTS: 1075

EXIT INVERTS: 1069

EMERGENCY DRAWDOWN FACILITIES: 16-inch diameter gate valve located at the bottom of a manhole along the embankment crest several feet to the left of the spillway

HYDROMETEOROLOGICAL GAGES

TYPE: None

LOCATION: -

RECORDS: -

MAXIMUM NON-DAMAGING DISCHARGE: Not known

APPENDIX B

CHECK LIST - VISUAL INSPECTION

CHECK LIST
VISUAL INSPECTION
PHASE 1

NAME OF DAM Twin Lakes No. 1 Dam STATE Pennsylvania COUNTY Westmoreland

NDI# PA - 487 PENNDER# 65-42

TYPE OF DAM Earth - rockfill SIZE Small HAZARD CATAGORY High

DATE(S) INSPECTION 13 December 1978 WEATHER Windy and cold TEMPERATURE 30° @ 9:00 a.m.

POOL ELEVATION AT TIME OF INSPECTION 1094.1 M.S.L.

TAILWATER AT TIME OF INSPECTION N/A M.S.L

INSPECTION PERSONNEL	OWNER REPRESENTATIVES	OTHERS
B. M. Mihalcin	Adrian Horvath	
D. L. Bonk		
S. R. Michalski		
W. J. Veon		

RECORDED BY D. L. Bonk

EMBANKMENT

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA - 487
SURFACE CRACKS	None observed.
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None observed.
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	None observed.
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	The embankment is well aligned from abutment to abutment. Differential settlements across the crest were measured to be less than one foot. The measured low spot of the embankment crest is at elevation 1097.7 or 0.3 feet below the top of dam design elevation at 1098.0 feet.
RIPRAP FAILURES	None observed.
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	Good condition.

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA - 487
DAMP AREAS IRREGULAR VEGETATION (LUSH OR DEAD PLANTS)	Several damp areas were observed across the downstream toe of the embankment both to the left and right of the principal spillway. A lack of grass and snow cover made these areas easily detectable. The wet spots appear to be more prevalent along the right side of the embankment toe, that is, to the right of the principal spillway located near the center of structure.
ANY NOTICEABLE SEEPAGE	No measureable flow could be detected at any of the observed wet areas. Most of the damp spots are located along a narrow flat area immediately beyond the toe between the toe and a roadway several feet downstream. However, several wet spots were located on the downstream face to the right of the spillway but are actually only a few feet from the base of the toe.
STAFF GAGE AND RECORDER	None observed.
DRAINS	The toe seepage drain pipe discharges into the base of the spillway channel directly opposite the outlet conduit discharge.

OUTLET WORKS

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA - 487
INTAKE STRUCTURE	Submerged.
OUTLET CONDUIT (CRACKING AND SPALLING OF CONCRETE SURFACES)	The blowoff conduit projects out of the left spillway wingwall near the downstream end of the spillway channel. This was the only portion of the outlet conduit that was visually observed by the field team. The conduit was not noticeably obstructed and showed no signs of unusual or advanced weathering.
OUTLET STRUCTURE	The old masonry gate house located to the left of the spillway at the base of the downstream toe is in good condition. It currently serves no function relative to the present facility. The present valve system is located at the base of a concrete chamber accessible via a manhole located along the embankment crest several feet to the left of the spillway left wingwall.
OUTLET CHANNEL	The outlet conduit discharges into the emergency spillway channel several feet beyond the toe (see "Discharge Channel," Sheet 5 of 8).
GATE(S) AND OPERATIONAL EQUIPMENT	The only operable device associated with the outlet works at this facility is a gate valve on the blowoff conduit. The concrete chamber where the valve is located was partially flooded on the day of the inspection and the valve itself submerged. An extension stem has been installed so that the valve can be operated practically from atop the embankment crest without entering the flooded chamber. The valve was opened and
	closed in the presence of the field team and proved to be functioning properly.

EMERGENCY SPILLWAY

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA - 487
TYPE AND CONDITION	Concrete channel with no weir located near the center of the embankment. At the crest, the channel is approximately 80 feet in width.
APPROACH CHANNEL	Sloping concrete channel. No signs of weathering were observed.
SPILLWAY CHANNEL AND SIDEWALLS	No signs of concrete weathering were observed. The channel and sidewalls are in excellent condition.
STILLING BASIN PLUNGE POOL	None. Flow through the emergency spillway is discharged directly into the stream below.
DISCHARGE CHANNEL	Fifteen to 20-foot wide, trapezoidal-shaped, unlined channel with approximately 3-foot high sidewalls.
BRIDGE AND PIERS	Immediately downstream of the spillway channel, a two-laned roadway bridge spans the stream. The bridge currently appears to be in a state of disrepair and could possibly be destroyed by the discharge of a major flood.
EMERGENCY GATES	None.

SERVICE SPILLWAY

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA - 487
TYPE AND CONDITION	N/A.
APPROACH CHANNEL	N/A.
OUTLET STRUCTURE	N/A.
DISCHARGE CHANNEL	N/A.

INSTRUMENTATION

NDI# PA - 487

ITEM	OBSERVATIONS AND/OR REMARKS
MONUMENTATION SURVEYS	None.
OBSERVATION WELLS	None.
WEIRS	None.
PIEZOMETERS	None.
OTHERS	None.

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA - 487
SLOPES: RESERVOIR	Steep and forested.
SEDIMENTATION	None observed.
DOWNSTREAM CHANNEL (OBSTRUCTIONS, DEBRIS, ETC.)	The channel downstream of the emergency spillway is a 15- to 20-foot wide, trapezoidal-shaped, unlined stream with approximately 3-foot high sidewalls. The stream is partially obstructed along the first several hundred feet by debris as well as overhanging and fallen trees.
SLOPES: CHANNEL VALLEY	The stream cuts through a valley with moderate lower slopes used primarily as pastureland and steep, forested upper slopes.
APPROXIMATE NUMBER OF HOMES AND POPULATION	Approximately one-half dozen homes could be affected by minor flooding due to their close proximity to the stream. A major flood could possibly endanger the lives of those persons residing in a trailer park near the community of Luxor approximately 1/2 mile downstream. It is estimated that more than 100 persons could be affected by a major flood from an embankment breach.

APPENDIX C

HYDROLOGY AND HYDRAULICS

PREFACE

The modified HEC-1 program is capable of performing two basic types of hydrologic analyses: (1) the evaluation of the overtopping potential of the dam; and (2) the estimation of the downstream hydrologic-hydraulic consequences resulting from assumed structural failures of the dam. Briefly, the computational procedures typically used in the dam overtopping analysis are as follows:

a. Development of an inflow hydrograph(s) to the reservoir.

b. Routing of the inflow hydrograph(s) through the reservoir to determine if the event(s) analyzed would overtop the dam.

c. Routing of the outflow hydrograph(s) from the reservoir to desired downstream locations. The results provide the peak discharge(s), time(s) of the peak discharge(s), and the maximum stage(s) of each routed hydrograph at the downstream end of each reach.

The evaluation of the hydrologic-hydraulic consequences resulting from an assumed structural failure (breach) of the dam is typically performed as outlined below.

a. Development of an inflow hydrograph(s) to the reservoir.

b. Routing of the inflow hydrograph(s) through the reservoir.

c. Development of a failure hydrograph(s) based on specific breach criteria and normal reservoir outflow.

d. Routing of the failure hydrograph(s) to desired downstream locations. The results provide estimates of the peak discharge(s), time(s) to peak, and maximum water surface elevation(s) of the failure hydrograph(s) for each location.

SUBJECT	DAM SAFETY INSPECTION			
	TWIN LAKES Nº 1 DAM			
BY WJV	DATE 4-2-79	PROJ. NO. 78-617-497		
CHKD. BY DLB	DATE 4-13-79	SHEET NO. 1	OF 9	

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

DAM STATISTICS

HEIGHT OF DAM ≈ 31 FEET (FIELD MEASURED)

MAXIMUM POOL STORAGE CAPACITY ≈ 470 AC-FT [OBTAINED FROM HEC-1 OUTPUT]
@ TOP OF DAM

NORMAL POOL STORAGE CAPACITY ≈ 340 AC-FT (SEE NOTE 1)

DRAINAGE AREA ≈ 1.51 SQ.MI. (LOCAL) [PLANIMETERED OFF USGS 7.5 MINUTE SERIES QUAD, LATROBE, PA.]
1.89 SQ MI. (TOTAL)

NOTE 1: STORAGE CAPACITY VALUE WAS OBTAINED FROM "DAMS, RESERVOIRS, AND NATURAL LAKES", WATER RESOURCES BULLETIN Nº 5, COMMONWEALTH OF PENNSYLVANIA, DEPARTMENT OF FORESTS AND WATERS, HARRISBURG, PA., 1970. THE REPORTED VALUE WAS 110 MILLION GALLONS. THIS VALUE WAS ALSO INDICATED ON FIGURE 2, APPENDIX F.

DAM CLASSIFICATION

DAM SIZE - SMALL (REF 1, TABLE 1)

HAZARD CLASSIFICATION - HIGH (FIELD OBSERVATION)

REQUIRED SDF - 1/2 PMF TO PMF (REF 1, TABLE 3)

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES Nº 1 DAM

BY WJV DATE 4-2-79 PROJ. NO. 78-617-497
CHKD. BY DLB DATE 4-13-79 SHEET NO. 2 OF 8

gai CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

HYDROGRAPH PARAMETERS

LENGTH OF LONGEST WATERCOURSE (L) ≈ 1.36 MI. (SEE NOTE 2)

L_{CA} ≈ 0.81 MI [MEASURED ALONG THE LONGEST WATERCOURSE FROM THE DAM CREST TO THE CENTROID OF THE REPRESENTATIVE SUB-BASIN]

NOTE 2: THREE INDEPENDENT STREAMS (BESIDES THE SMALL STREAM GENERATED BY THE OUTFLOWS OF THE UPSTREAM UPPER DONOHOE DAM) DRAIN THE LOCAL 1.51 SQ.MI. BASIN. EACH OF THE STREAMS ENTERS THE RESERVOIR AT A DISTINCTLY DIFFERENT POINT AND COLLECTS RUNOFF FROM ABOUT 1/3 OF THE LOCAL AREA (SEE REGIONAL VICINITY MAP, APPENDIX G). THE L AND LCA PARAMETERS ARE ALSO APPROXIMATELY THE SAME FOR EACH STREAM SUB-BASIN. THEREFORE, INSTEAD OF CONSIDERING A SEPARATE LOCAL RESERVOIR INFLOW HYDROGRAPH FOR EACH OF THE STREAMS, ONLY ONE LARGER LOCAL INFLOW HYDROGRAPH WILL BE COMPUTED IN THE HEC-1 ANALYSIS. THIS IS DONE UNDER THE ASSUMPTION THAT A HYDROGRAPH GENERATED BY APPLYING A RAINFALL DISTRIBUTION TO A NUMBER (3) OF SEPARATE BUT QUANTITATIVELY EQUAL UNIT HYDROGRAPHS AND ADDING THE RESULTS CAN BE APPROXIMATED BY APPLYING THE RAINFALL DISTRIBUTION TO A UNIT HYDROGRAPH WHICH IS A NUMBER (3) TIMES LARGER THAN ANY ONE OF THE SEPERATE BUT EQUAL UNIT GRAPHS. THE LARGE UNIT GRAPH TO BE COMPUTED BY HEC-1 WILL BE BASED ON THE ENTIRE LOCAL DRAINAGE AREA AS WELL AS THE L AND LCA VALUES ABOVE WHICH WERE MEASURED FOR THE SUB-BASIN WHICH CONTAINED THE LARGEST OF THE THREE STREAMS (SEE REGIONAL VICINITY MAP, REPRESENTATIVE SUB-BASIN). (VALUES OF L AND LCA WERE MEASURED FROM THE USGS 7.5 MINUTE LATROBE, PA QUAD)

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES No 1 DAM
BY WJV DATE 4-3-79 PROJ. NO. 78-617-487
CHKD. BY DLB DATE 4-13-79 SHEET NO. 3 OF 8

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

$C_t \approx 1.6$
$C_p \approx 0.45$

[SUPPLIED BY COE; ZONE 24, OHIO RIVER BASIN]

$\therefore \; t_p$ = SNYDER'S STANDARD LAG = $1.6\,(L \times L_{CA})^{0.3}$

$$t_p = 1.6\,[(1.36)\times(0.81)]^{0.3} \approx 1.65 \text{ HR}$$

RESERVOIR SURFACE AREAS

SURFACE AREA (SA) @ NORMAL POOL EL. 1094.0 ≈ 33.3 ACRES

NOTE 3: SURFACE AREA VALUES WERE OBTAINED FROM FIGURE 2, APPENDIX F, BY PLANIMETERING THE AREAS BETWEEN THE RESPECTIVE CONTOUR LINES AND THE DAM CREST. ACTUAL NORMAL POOL ELEVATION OF 1094.0 WAS OBTAINED FROM A COMBINATION OF FIGURES 2 AND 6, APPENDIX F. (CONSTRUCTION DRAWINGS IN APPENDIX F ARE 3 FT LOWER THAN ACTUAL ELEVATIONS => SEE NOTES ON FIG. 2)

SA @ EL. 1100 FT ≈ 42.4 ACRES

RATE OF AREA CHANGE PER FOOT OF RESERVOIR RISE:

$$\Delta A / \Delta H = (42.4 - 33.3) \text{ ACRES} / (1100.0 - 1094.0) \text{ FEET}$$

$$\Delta A / \Delta H \approx 1.5 \text{ ACRES/FOOT}$$

SA @ TOP OF DAM EL. 1097.7 ≈ (1.5 A/FT)(1097.7 - 1094.0) + 33.3 A
(LOW TOP OF DAM ELEVATION => FIELD MEASURED)

SA @ EL. 1097.7 ≈ 33.9 ACRES

SUBJECT	DAM SAFETY INSPECTION TWIN LAKES No 1 DAM				
BY	WJV	DATE	4-3-79	PROJ. NO.	79-617-437
CHKD. BY	DLB	DATE	4-13-79	SHEET NO.	4 OF 8

GAI CONSULTANTS, I

Engineers • Geologists • Planners
Environmental Specialists

RESERVOIR ELEVATION @ "O" STORAGE

NORMAL POOL VOLUME ≈ 1/3 HA ≈ 340 AC-FT (CONIC METHOD)

SA @ NORMAL POOL EL. 1094.0 ≈ 33.3 ACRES

$\therefore \; H = \frac{3V}{A} \approx 3(340 \text{ AC-FT}) / (33.3 \text{ ACRES}) \approx 30.6 \text{ FT}$

ZERO VOLUME ELEVATION ≈ 1094.0 - 30.6 ≈ 1063.4 FT

NOTE 4: ALTHOUGH THE ACTUAL MINIMUM RESERVOIR ELEVATION @ "O" STORAGE IS ≈ 1072.0 FT. (FIG. 7, APPENDIX F), IN ORDER TO COMPUTE A STORAGE-DISCHARGE RELATIONSHIP AND STILL MAINTAIN A STORAGE OF 340 AC-FT @ NORMAL POOL, THE ABOVE CALCULATED "O" STORAGE ELEVATION OF 1063.4 MUST BE INPUT INTO THE HEC-1 PROGRAM.

STORAGE - ELEVATION RELATIONSHIP

COMPUTED INTERNALLY BY THE HEC-1 PROGRAM, BASED ON GIVEN SURFACE AREA VS ELEVATION INFORMATION. (SEE SUMMARY INPUT/OUTPUT SHEETS)

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES No 1 DAM

BY WJV DATE 4-3-79 PROJ. NO. 79-617-487
CHKD. BY DLB DATE 4-13-79 SHEET NO. 5 OF 8

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

PMP CALCULATIONS

- APPROXIMATE RAINFALL INDEX = 24 INCHES (REF. 3, FIG. 1)
(CORRESPONDING TO A DURATION OF 24 HRS AND A DRAINAGE AREA OF 200 SQ. MI LOCATED IN SOUTHWESTERN PENNSYLVANIA)

- DEPTH-AREA-DURATION ZONE #7 (REF 3, FIG 1)

- ALTHOUGH THE LOCAL DRAINAGE AREA ≈ 1.51 SQ. MI., THE AREA OVER WHICH THE PMP WILL BE CENTERED IS THE TOTAL 1.89 SQ. MI. BASIN AREA ⇒ ASSUME THAT DATA CORRESPONDING TO A 10 SQ. MI. AREA IS REPRESENTATIVE OF THIS BASIN:

DURATION (HR)	PERCENT OF INDEX RAINFALL (%)
6	102.0
12	120.0
24	130.0

NOTE 5: A 24-HR RATHER THAN A 48-HR DURATION IS USED SO THAT A TIME STEP OF 5-MINUTES CAN BE USED IN THE HEC-1 PROGRAM

- HOP BROOK FACTOR (ADJUSTMENT FOR BASIN SHAPE AS WELL AS FOR THE LESSER LIKELIHOOD OF A SEVERE STORM CENTERING OVER A SMALLER BASIN) CORRESPONDING TO A DA = 1.89 SQ. MI. (< 10 SQ. MI.) ≈ 0.80 (REF 4, PG 48).

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES Nº 1 DAM

BY WJV DATE 4-3-79 PROJ. NO. 78-617-437

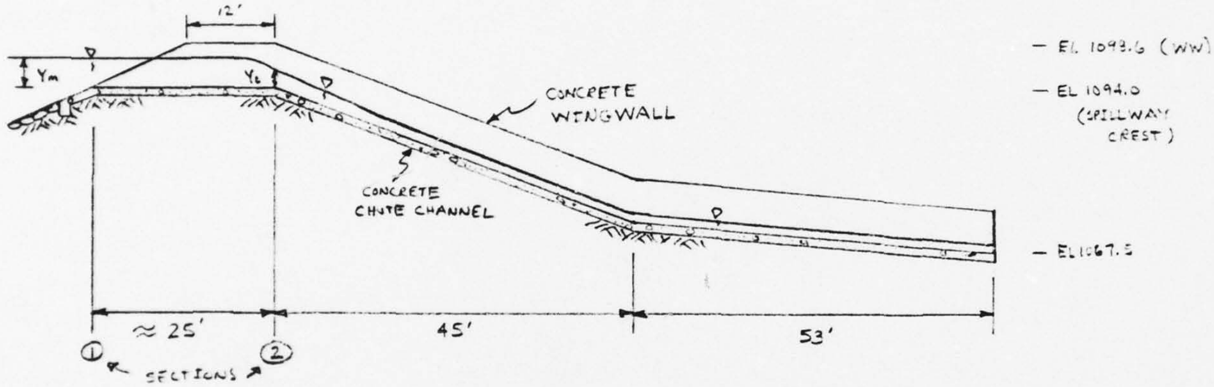
CHKD. BY DLB DATE 4-13-79 SHEET NO. 6 OF 8

GAI CONSULTANTS, INC.

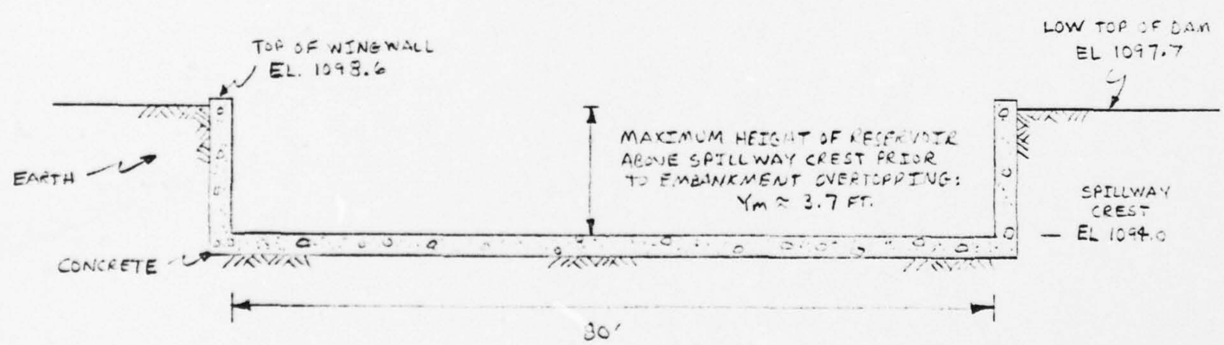
Engineers • Geologists • Planners
Environmental Specialists

SPILLWAY CAPACITY

- SPILLWAY PROFILE : (NOT TO SCALE)



- SPILLWAY CREST SECTION : (NOT TO SCALE)



- ASSUMING THAT THE WATER SURFACE PROFILE PASSES THROUGH CRITICAL DEPTH @ SECTION ② : ENERGY BALANCE BETWEEN ① AND ② ⇒

$$Y_m + \cancel{v_1^2/2g}^{\,0} + z_1 = Y_c + v_c^2/2g + z_2 + \cancel{H_L}^{\,0}$$ (REF 7, PG. 4

WHERE v_1 = RESERVOIR VELOCITY ≈ 0 FPS.,
z_1 = ELEVATION @ ① IN FT.,
v_c = CRITICAL VELOCITY @ ② IN FPS.,

GAI CONSULTANTS, I
Engineers • Geologists • Planners
Environmental Specialists

Z_2 = ELEVATION @ ② IN FT., AND
H_L = HEAD LOSS BETWEEN ① AND ② ≈ 0

- SINCE $Z_1 - Z_2 \approx 0$ (SECTIONS ① AND ② ARE CLOSE TOGETHER)

$$Y_m = Y_c + v_c^2/2g \qquad w/\ Y_m = 3.7\text{ FT}$$

- FOR CRITICAL DEPTH IN A RECTANGULAR SECTION:

$$v_c^2/2g = Y_c/2 \qquad \text{(REF 7, PG. 55)}$$

$$\therefore \quad Y_m = 3.7\text{ FT} = Y_c + Y_c/2 = 3/2\ Y_c$$

$$Y_c \approx 2.47\text{ FT}$$

- CRITICAL AREA = $A_c \approx (80\text{ FT})(Y_c) = (80\text{ FT})(2.47\text{ FT}) \approx 197.6\text{ FT}^2$

- CRITICAL VELOCITY $\Rightarrow$ $v_c = \sqrt{g Y_c}$ (FROM ABOVE)

$$v_c = \sqrt{g\,(2.47\text{ FT})}$$

$$v_c \approx 8.92\text{ FPS}$$

$\therefore$ SPILLWAY CAPACITY = $Q = A_c v_c = (197.6\text{ FT}^2)(8.92\text{ FPS})$

$$Q \approx 1760\text{ CFS}$$

NOTE 6: IF DAM CREST WAS LEVEL @ DESIGN ELEVATION 1098.0 FT
$\Rightarrow Y_m = 4 = 3/2\ Y_c \Rightarrow Y_c \approx 2.67\text{ FT}$; $v_c^2/2g \approx 1.33\text{ FT} \Rightarrow$
$v_c \approx 9.25\text{ FPS}$; $Q = A_c v_c \approx [80(2.67)][9.25] \approx 1980\text{ CFS}$

SUBJECT DAM SAFETY INSPECTION

TWIN LAKES Nº 1 DAM

BY WJV DATE 4-3-79 PROJ. NO. 79-617-437

CHKD. BY DLB DATE 4-13-79 SHEET NO. 8 OF 8

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

SPILLWAY RATING CURVE

COMPUTED INTERNALLY BY HEC-1 VIA THE TRAPEZOIDAL RATING CURVE ROUTINE, BASED ON THE SPILLWAY GEOMETRY AS PRESENTED ON SHEET 6. THE TRAPEZOIDAL ROUTINE CALCULATES CRITICAL CONTROL DISCHARGES IN A WAY SIMILAR TO THAT OUTLINED ON SHEETS 6 AND 7. (SEE SUMMARY INPUT / OUTPUT SHEETS).

DAM EMBANKMENT RATING CURVE

- COMPUTED INTERNALLY BY HEC-1 VIA THE ASSUMPTION THAT CRITICAL DEPTH OCCURS ON THE CREST (WHEN OVERTOPPED), W/ THE CREST PROFILE REPRESENTED BY A SERIES OF TRAPEZOIDS. (SEE SUMMARY INPUT / OUTPUT SHEETS FOR RATING INFORMATION).

- INPUT INFORMATION: (BASED ON FIELD MEASUREMENTS)

	RESERVOIR ELEVATION (FT)	DEPTH OF WATER ABOVE CREST (FT)	LENGTH OF CREST INUNDATED (FT)
TOP OF DAM -	1097.7	0	300
	1097.8	0.1	430
	1098.0	0.3	520
	1098.1	0.4	740
	1098.5	0.8	800
	1099.0	1.3	830
	1099.5	1.8	860
	1100.0	2.3	890

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES No 1 DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-487

CHKD. BY DLB DATE 4-21-79 SHEET NO. A OF L

GAI CONSULTANTS.
Engineers • Geologists • Planne
Environmental Specialists

OVERTOPPING

SUMMARY INPUT/OUTPUT SHEETS

DAM SAFETY INSPECTION
UPPER DONOHOE + TWIN LAKES NO 1 DAMS ****** OVERTOPPING ANALYSIS ******
5-MINUTE TIME STEPS AND 24-HOUR STORM DURATION

JOB SPECIFICATION

NQ	NHR	NMIN	IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
288	0	5	0	0	0	0	0	0	0

JOPER	NWT	LROPT	TRACE
5	0	0	0

MULTI-PLAN ANALYSES TO BE PERFORMED
NPLAN= 1 NRTIO= 6 LRTIO= 1
RTIOS= .30 .40 .50 .60 .70 1.00

********** ********** ********** ********** **********

SUB-AREA RUNOFF COMPUTATION

INFLOW INTO ARTIFICIAL RESERVOIR CAUSED BY RAILROAD EMBANKMENT AND CULVERT

NOTE: REFER TO APPENDIX C-1 FOR INFORMATION CONCERNING UPPER DONOHOE DAM.

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
1	0	0	0	0	0	1	0	0

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	1	.22	0.00	1.89	0.00	0.000	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	24.00	102.00	120.00	130.00	0.00	0.00	0.00

TRSPC COMPUTED BY THE PROGRAM IS .800

LOSS DATA

LROPT	STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0	0.00	0.00	1.00	0.00	0.00	1.00	1.00	.05	0.00	0.00

UNIT HYDROGRAPH DATA
TP= 1.00 CP= .45 NTA= 0

RECESSION DATA
STRTQ= -1.50 QRCSN= -.05 RTIOR= 2.00
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC=12.50 AND R=19.13 INTERVALS

SUBJECT DAM SAFETY INSPECTION

TWIN LAKES NO 1 DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-497

CHKD. BY DLB DATE 4-21-79 SHEET NO. B OF L

gai CONSULTANTS
Engineers • Geologists • Planners
Environmental Specialists

UNIT HYDROGRAPH100 END-OF-PERIOD ORDINATES, LAG= 1.01 HOURS, CP= .45 VOL= .99

1.	5.	11.	17.	25.	33.	41.	49.	55.	60.
64.	65.	65.	62.	59.	56.	53.	50.	48.	45.
43.	41.	39.	37.	35.	33.	31.	30.	28.	27.
25.	24.	23.	22.	21.	20.	19.	18.	17.	16.
15.	14.	14.	13.	12.	12.	11.	10.	10.	9.
9.	8.	8.	8.	7.	7.	7.	6.	6.	6.
5.	5.	5.	5.	4.	4.	4.	4.	3.	3.
3.	3.	3.	3.	3.	2.	2.	2.	2.	2.
2.	2.	2.	2.	2.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.

0

END-OF-PERIOD FLOW

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
									SUM	24.96	23.08	1.88	38450.
										(634.)	(586.)	(48.)	(1088.78)

INFLOWS US OF RAILROAD EMBANKMENT CULVERT

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	643.	396.	133.	133.	38435.
CMS	18.	11.	4.	4.	1088.
INCHES		16.75	22.57	22.57	22.57
MM		425.33	573.32	573.32	573.32
AC-FT		196.	265.	265.	265.
THOUS CU M		242.	327.	327.	327.

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	321.	198.	67.	67.	19217.
CMS	9.	6.	2.	2.	544.
INCHES		8.37	11.29	11.29	11.29
MM		212.66	286.66	286.66	286.66
AC-FT		98.	132.	132.	132.
THOUS CU M		121.	163.	163.	163.

0.6 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	386.	238.	80.	80.	23061.
CMS	11.	7.	2.	2.	653.
INCHES		10.05	13.54	13.54	13.54
MM		255.20	343.99	343.99	343.99
AC-FT		118.	159.	159.	159.
THOUS CU M		145.	196.	196.	196.

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES № 1 DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-497

CHKD. BY DLB DATE 4-21-79 SHEET NO. C OF L

GAI CONSULTANTS.
Engineers • Geologists • Planners
Environmental Specialists

HYDROGRAPH ROUTING

ROUTE THROUGH ARTIFICIAL RESERVOIR AND INTO UPPER DONOHOE DAM RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
101	1	0	0	0	0	1	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP		LSTR
0.0	0.000	0.00	1	1	0	0		0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	-1135.	-1

STAGE	1135.00	1136.00	1137.00	1138.00	1139.00	1140.00	1141.00	1142.00	1143.00
	1145.00	1146.00	1147.00	1147.40	1148.60	1152.30	1156.30	1160.70	1165.30
FLOW	0.00	10.00	20.00	40.00	60.00	70.00	80.00	85.00	90.00
	105.00	110.00	115.00	117.00	120.00	130.00	140.00	150.00	160.00

SURFACE AREA=	0.	3.	10.	27.
CAPACITY=	0.	5.	126.	480.
ELEVATION=	1135.	1140.	1160.	1180.

CREL	SPWID	COQW	EXPW	ELEVL	COOL	CAREA	EXPL
1180.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

DAM DATA

TOPEL	COQD	EXPD	DAMWID
1180.0	0.0	0.0	0.

OUTFLOWS FROM RAILROAD EMBANKMENT CULVERT

PEAK OUTFLOW IS 152. AT TIME 20.33 HOURS

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	152.	150.	73.	73.	21088.
CMS	4.	4.	2.	2.	597.
INCHES		6.35	12.38	12.38	12.38
MM		161.29	314.56	314.56	314.56
AC-FT		74.	145.	145.	145.
THOUS CU M		92.	179.	179.	179.

PEAK OUTFLOW IS 125. AT TIME 19.42 HOURS

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	125.	122.	55.	55.	15846.
CMS	4.	3.	2.	2.	449.
INCHES		5.15	9.31	9.31	9.31
MM		130.81	236.36	236.36	236.36
AC-FT		60.	109.	109.	109.
THOUS CU M		74.	135.	135.	135.

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES NR 1 DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-437

CHKD. BY DLB DATE 4-21-79 SHEET NO. D OF L

gai CONSULTANTS
Engineers • Geologists • Planners
Environmental Specialists

0.6 PMF

OUTFLOW FROM RR CULVERT

PEAK OUTFLOW IS 132. AT TIME 19.67 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	132.	129.	60.	60.	17182.
CMS	4.	4.	2.	2.	487.
INCHES		5.46	10.09	10.09	10.09
MM		138.60	256.29	256.29	256.29
AC-FT		64.	118.	118.	118.
THOUS CU M		79.	146.	146.	146.

SUB-AREA RUNOFF COMPUTATION

LOCAL INFLOW INTO UPPER DONOHOE DAM RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
2	0	0	0	0	0	1	0	0

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	1	.16	0.00	1.89	0.00	0.000	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	24.00	102.00	120.00	130.00	0.00	0.00	0.00

TRSPC COMPUTED BY THE PROGRAM IS .800

LOSS DATA

LROPT	STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0	0.00	0.00	1.00	0.00	0.00	1.00	1.00	.05	0.00	0.00

UNIT HYDROGRAPH DATA
TP= .57 CP= .45 NTA= 0

RECESSION DATA
STRTQ= -1.50 QRCSN= -.05 RTIOR= 2.00

APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 7.18 AND R=10.95 INTERVALS

UNIT HYDROGRAPH 62 END-OF-PERIOD ORDINATES, LAG= .57 HOURS, CP= .45 VOL= 1.00

4.	15.	30.	48.	64.	76.	82.	80.	73.	67.
61.	56.	51.	46.	42.	39.	35.	32.	29.	27.
24.	22.	20.	19.	17.	15.	14.	13.	12.	11.
10.	9.	8.	7.	7.	6.	6.	5.	5.	4.
4.	4.	3.	3.	3.	2.	2.	2.	2.	2.
2.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.								

SUBJECT DAM SAFETY INSPECTION

TWIN LAKES No 1 DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-497

CHKD. BY DLB DATE 4-21-79 SHEET NO. E OF L

CONSULTANTS

Engineers • Geologists • Plann
Environmental Specialists

0 MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	END-OF-PERIOD FLOW COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
									SUM	24.96	23.08	1.88	28250.
										(634.)(	586.)(	48.)(	799.95)

LOCAL UPPER DONOHOE RESERVOIR INFLOW

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	626.	309.	98.	98.	28239.
CMS	18.	9.	3.	3.	800.
INCHES		17.97	22.80	22.80	22.80
MM		456.35	579.19	579.19	579.19
AC-FT		153.	194.	194.	194.
THOUS CU M		189.	240.	240.	240.

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	313.	155.	49.	49.	14119.
CMS	9.	4.	1.	1.	400.
INCHES		8.98	11.40	11.40	11.40
MM		228.18	289.59	289.59	289.59
AC-FT		77.	97.	97.	97.
THOUS CU M		95.	120.	120.	120.

0.6 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	375.	185.	59.	59.	16943.
CMS	11.	5.	2.	2.	480.
INCHES		10.78	13.68	13.68	13.68
MM		273.81	347.51	347.51	347.51
AC-FT		92.	117.	117.	117.
THOUS CU M		113.	144.	144.	144.

COMBINE HYDROGRAPHS

COMBINE RAILROAD EMBANKMENT OUTFLOWS W/ LOCAL INFLOWS FOR TOTAL INFLOW

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
2	2	0	0	0	0	1	0	0

TOTAL UPPER DONOHOE RESERVOIR INFLOWS

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	749.	431.	171.	171.	49327.
CMS	21.	12.	5.	5.	1397.
INCHES		10.54	16.77	16.77	16.77
MM		267.83	425.98	425.98	425.98
AC-FT		214.	340.	340.	340.
THOUS CU M		263.	419.	419.	419.

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	410.	253.	104.	104.	29965.
CMS	12.	7.	3.	3.	849.
INCHES		6.20	10.19	10.19	10.19
MM		157.45	258.78	258.78	258.78
AC-FT		126.	206.	206.	206.
THOUS CU M		155.	255.	255.	255.

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES Nº 1 DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-497

CHKD. BY DLB DATE 4-21-79 SHEET NO. F OF L

GAI CONSULTANTS
Engineers • Geologists • Plann
Environmental Specialists

0.6 PMF

TOTAL UPPER DONOHOE RESERVOIR INFLOW

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	481.	290.	118.	118.	34125.
CMS	14.	8.	3.	3.	966.
INCHES		7.10	11.60	11.60	11.60
MM		180.23	294.70	294.70	294.70
AC-FT		144.	235.	235.	235.
THOUS CU M		177.	290.	290.	290.

HYDROGRAPH ROUTING

ROUTE TOTAL INFLOW HYDROGRAPH THROUGH UPPER DONOHOE DAM RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
202	1	0	0	0	0	1	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	-1126.	1

SURFACE AREA=	0.	20.	23.	24.	29.
CAPACITY=	0.	215.	273.	304.	437.
ELEVATION=	1094.	1126.	1129.	1130.	1135.

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
1126.0	20.0	0.0	0.0	0.0	0.0	0.0	0.0

IABCOA	ISPITW	ISPCTW
10	0	0

SS	NGATES	DESHD	APEL	APWID	APLOSS	POPTH
0.00	1	2.7	0.0	0.0	0.0	0.0

1093.90	0.	0.	0.0	0.	0.	0.0	0.	0.	0.
1126.00	215.	0.	0.0	0.	0.	0.0	0.	0.	0.
1128.70	273.	267.	1.8	267.	0.	0.0	0.	267.	267.
1130.00	304.	474.	2.7	474.	0.	0.0	0.	474.	474.
1135.00	437.	1518.	6.0	1518.	0.	0.0	0.	1518.	1518.

DAM DATA

TOPEL	COQD	EXPD	DAMWID
1128.7	0.0	0.0	0.

CREST LENGTH AT OR BELOW	0.	175.	225.	325.	375.	420.	500.	521.	526.	531.
ELEVATION	1128.7	1128.9	1129.0	1129.1	1129.2	1129.3	1129.5	1129.7	1130.2	1130.7

SUBJECT DAM SAFETY INSPECTION

TWIN LAKES No 1 DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-437

CHKD. BY DLB DATE 4-21-79 SHEET NO. 6 OF L

GAI CONSULTANTS. Engineers • Geologists • Plann- Environmental Specialists

UPPER DONOHOE OUTFLOWS

OVERTOPPING @ ≈0.54 PMF

PMF

PEAK OUTFLOW IS 640. AT TIME 16.58 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	640.	389.	151.	151.	43404.
CMS	18.	11.	4.	4.	1229.
INCHES		9.52	14.76	14.76	14.76
MM		241.72	374.84	374.84	374.84
AC-FT		193.	299.	299.	299.
THOUS CU M		238.	369.	369.	369.

0.5 PMF

PEAK OUTFLOW IS 250. AT TIME 18.33 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	250.	219.	88.	88.	25458.
CMS	7.	6.	3.	3.	721.
INCHES		5.37	8.66	8.66	8.66
MM		136.40	219.85	219.85	219.85
AC-FT		109.	175.	175.	175.
THOUS CU M		134.	216.	216.	216.

0.6 PMF

PEAK OUTFLOW IS 300. AT TIME 18.08 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	300.	251.	101.	101.	29140.
CMS	9.	7.	3.	3.	825.
INCHES		6.14	9.91	9.91	9.91
MM		156.07	251.65	251.65	251.65
AC-FT		124.	201.	201.	201.
THOUS CU M		154.	248.	248.	248.

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES No 1 DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-487

CHKD. BY DLB DATE 4-21-79 SHEET NO. H OF L

GAI CONSULTANTS, Engineers • Geologists • Planners, Environmental Specialists

SUB-AREA RUNOFF COMPUTATION

LOCAL INFLOW INTO TWIN LAKES NO 1 DAM RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
3	0	0	0	0	0	1	0	0

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	1	1.51	0.00	1.89	0.00	0.000	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	24.00	102.00	120.00	130.00	0.00	0.00	0.00

TRSPC COMPUTED BY THE PROGRAM IS .800

LOSS DATA

LROPT	STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0	0.00	0.00	1.00	0.00	0.00	1.00	1.00	.05	0.00	0.00

UNIT HYDROGRAPH DATA
TP= 1.65 CP= .45 NTA= 0

RECESSION DATA
STRTQ= -1.50 QRCSN= -.05 RTIOR= 2.00

APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC=20.42 AND R=31.63 INTERVALS

UNIT HYDROGRAPH100 END-OF-PERIOD ORDINATES, LAG= 1.65 HOURS, CP= .45 VOL= .94

3.	11.	22.	36.	51.	68.	87.	106.	126.	148.
169.	189.	208.	224.	238.	250.	260.	267.	272.	(273.)
269.	262.	254.	246.	238.	231.	223.	217.	210.	203.
197.	191.	185.	179.	174.	168.	163.	158.	153.	148.
144.	139.	135.	131.	126.	123.	119.	115.	111.	108.
105.	101.	98.	95.	92.	89.	87.	84.	81.	79.
76.	74.	72.	69.	67.	65.	63.	61.	59.	57.
56.	54.	52.	51.	49.	47.	46.	45.	43.	42.
41.	39.	38.	37.	36.	35.	34.	32.	31.	30.
30.	29.	28.	27.	26.	25.	24.	24.	23.	22.

0

END-OF-PERIOD FLOW

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
									SUM	24.96	23.08	1.88	247144.
										(634.)	(586.)	(48.)	(6998.34)

LOCAL TWIN LAKES No 1 RESERVOIR INFLOW

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3355.	2420.	858.	858.	246990.
CMS	95.	69.	24.	24.	6994.
INCHES		14.91	21.13	21.13	21.13
MM		378.71	536.78	536.78	536.78
AC-FT		1200.	1701.	1701.	1701.
THOUS CU M		1480.	2098.	2098.	2098.

PMF

SUBJECT DAM SAFETY INSPECTION

TWIN LAKES Nº 1 DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-497

CHKD. BY DLB DATE 4-21-79 SHEET NO. I OF L

GAI CONSULTANTS

Engineers • Geologists • Planne
Environmental Specialists

LOCAL TWIN LAKES Nº1 RESERVOIR INFLOWS

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1677.	1210.	429.	429.	123495.
CMS	47.	34.	12.	12.	3497.
INCHES		7.45	10.57	10.57	10.57
MM		189.35	268.39	268.39	268.39
AC-FT		600.	851.	851.	851.
THOUS CU M		740.	1049.	1049.	1049.

0.6 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2013.	1452.	515.	515.	148194.
CMS	57.	41.	15.	15.	4196.
INCHES		8.95	12.68	12.68	12.68
MM		227.23	322.07	322.07	322.07
AC-FT		720.	1021.	1021.	1021.
THOUS CU M		888.	1259.	1259.	1259.

COMBINE HYDROGRAPHS

COMBINE UPPER DONOHOE OUTFLOWS W/ LOCAL INFLOWS FOR TOTAL INFLOW

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
3	2	0	0	0	0	1	0	0

TOTAL TWIN LAKES Nº1 RESERVOIR INFLOWS

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3932.	2607.	1008.	1008.	290394.
CMS	111.	74.	29.	29.	8223.
INCHES		13.81	19.85	19.85	19.85
MM		350.89	504.22	504.22	504.22
AC-FT		1392.	2000.	2000.	2000.
THOUS CU M		1717.	2467.	2467.	2467.

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1913.	1421.	517.	517.	148953.
CMS	54.	40.	15.	15.	4218.
INCHES		6.99	10.18	10.18	10.18
MM		177.63	258.63	258.63	258.63
AC-FT		705.	1026.	1026.	1026.
THOUS CU M		869.	1265.	1265.	1265.

0.6 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2289.	1695.	616.	616.	177334.
CMS	65.	48.	17.	17.	5022.
INCHES		8.34	12.12	12.12	12.12
MM		211.88	307.91	307.91	307.91
AC-FT		840.	1221.	1221.	1221.
THOUS CU M		1037.	1506.	1506.	1506.

SUBJECT: DAM SAFETY INSPECTION
TWIN LAKES Nº 1 DAM

BY: WJV DATE: 4-20-79 PROJ. NO. 79-617-187
CHKD. BY: DLB DATE: 4-21-79 SHEET NO. J OF L

gai CONSULTANTS.
Engineers • Geologists • Planne
Environmental Specialists

PMF

HYDROGRAPH ROUTING

ROUTE TOTAL INFLOW HYDROGRAPH THROUGH TWIN LAKES NO 1 DAM RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
303	1	0	0	0	0	1	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	-1094.	1

SURFACE AREA=	0.	33.	39.	42.
CAPACITY=	0.	340.	473.	567.
ELEVATION=	1063.	1094.	1098.	1100.

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
1094.0	80.0	0.0	0.0	0.0	0.0	0.0	0.0

IABCOA	ISPITW	ISPCTW
10	0	0

SS	NGATES	DESHD	APEL	APWID	APLOSS	PDPTH
0.00	1	3.7	0.0	0.0	0.0	0.0

1063.40	0.	0.	0.0	0.	0.	0.0	0.	0.	0.
1094.00	340.	0.	0.0	0.	0.	0.0	0.	0.	0.
1097.70	473.	1742.	2.5	1742.	0.	0.0	0.	1742.	1742.
1100.00	567.	3577.	4.0	3577.	0.	0.0	0.	3577.	3577.

DAM DATA

TOPEL	COQD	EXPD	DAMWID
1097.7	0.0	0.0	0.

CREST LENGTH AT OR BELOW	300.	430.	520.	740.	800.	830.	860.	890.
ELEVATION	1097.7	1097.8	1098.0	1098.1	1098.5	1099.0	1099.5	1100.0

TWIN LAKES Nº 1 OUTFLOW

PEAK OUTFLOW IS 3911. AT TIME 17.25 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3911.	2754.	978.	978.	281709.
CMS	111.	78.	28.	28.	7977.
INCHES		13.55	19.26	19.26	19.26
MM		344.28	489.14	489.14	489.14
AC-FT		1366.	1940.	1940.	1940.
THOUS CU M		1684.	2393.	2393.	2393.

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES Nº 1 DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-497

CHKD. BY DLB DATE 4-21-79 SHEET NO. K OF L

GAI CONSULTANTS,
Engineers • Geologists • Planne
Environmental Specialists

TWIN LAKES Nº 1 OUTFLOWS

PEAK OUTFLOW IS 1721. AT TIME 18.50 HOURS

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1721.	1383.	500.	500.	144036.
CMS	49.	39.	14.	14.	4079.
INCHES		6.81	9.85	9.85	9.85
MM		172.85	250.09	250.09	250.09
AC-FT		686.	992.	992.	992.
THOUS CU M		846.	1224.	1224.	1224.

OVERTOPPING @ ≈ 0.51 PMF

PEAK OUTFLOW IS 2200. AT TIME 17.92 HOURS

0.6 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2200.	1650.	596.	596.	171602.
CMS	62.	47.	17.	17.	4859.
INCHES		8.12	11.73	11.73	11.73
MM		206.29	297.96	297.96	297.96
AC-FT		818.	1182.	1182.	1182.
THOUS CU M		1009.	1458.	1458.	1458.

SUMMARY OF DAM SAFETY ANALYSIS

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1135.00	1180.00	1180.00
STORAGE	0.	480.	480.
OUTFLOW	0.	190.	190.

RAILROAD EMBANKMENT

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.30	1144.68	0.00	21.	103.	0.00	18.92	0.00
.40	1147.68	0.00	34.	118.	0.00	19.17	0.00
.50	1150.53	0.00	50.	125.	0.00	19.42	0.00
.60	1153.15	0.00	68.	132.	0.00	19.67	0.00
.70	1155.53	0.00	86.	138.	0.00	19.83	0.00
1.00	1161.66	0.00	144.	152.	0.00	20.33	0.00

SUBJECT: DAM SAFETY INSPECTION

TWIN LAKES No 1 DAM

BY: WJV DATE: 4-20-79 PROJ. NO.: 79-617-497

CHKD. BY: DLB DATE: 4-21-79 SHEET NO.: L OF L

GAI CONSULTANTS,
Engineers • Geologists • Planne
Environmental Specialists

SUMMARY OF DAM SAFETY ANALYSIS

UPPER DONDHOE DAM

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1126.00	1126.00	1128.70
STORAGE	215.	215.	273.
OUTFLOW	0.	0.	267.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.30	1127.72	0.00	251.	170.	0.00	18.50	0.00
.40	1128.14	0.00	260.	211.	0.00	18.42	0.00
.50	1128.53	0.00	269.	250.	0.00	18.33	0.00
.60	1128.85	.15	277.	300.	2.08	18.08	0.00
.70	1129.03	.33	281.	380.	3.08	17.50	0.00
1.00	1129.30	.60	287.	640.	4.33	16.58	0.00

TWIN LAKES No 1 DAM

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1094.00	1094.00	1097.70
STORAGE	340.	340.	473.
OUTFLOW	0.	0.	1742.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.30	1096.24	0.00	418.	1055.	0.00	18.42	0.00
.40	1096.95	0.00	445.	1391.	0.00	18.50	0.00
.50	1097.65	0.00	471.	1721.	0.00	18.50	0.00
.60	1098.00	.30	485.	2200.	2.42	17.92	0.00
.70	1098.20	.50	493.	2675.	3.25	17.58	0.00
1.00	1098.57	.87	507.	3911.	5.00	17.25	0.00

LIST OF REFERENCES

1. "Recommended Guidelines for Safety Inspection of Dams," prepared by Department of the Army Office of the Chief of Engineers, Washington, D. C. (Appendix D).

2. "Unit Hydrograph Concepts and Calculations," by Corps of Engineers, Baltimore District (L-519).

3. "Seasonal Variation of Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Duration of 6, 12, 24, and 48 Hours," Hydrometeorological Report No. 33, prepared by J. T. Riedel, J. F. Appleby and R. W. Schloemer Hydrologic Service Division Hydrometeorological Section, U. S. Department of the Army, Corps of Engineers, Washington, D. C., April 1956.

4. Design of Small Dams, U. S. Department of the Interior, Bureau of Reclamation, Washington, D. C., 1973.

5. Handbook of Hydraulic, H. W. King and E. F. Brater, McGraw-Hill, Inc., New York, 1963.

6. Standard Handbook for Civil Engineers, F. S. Merritt McGraw-Hill, Inc., New York, 1968.

7. Open-Channel Hydraulics, V. T. Chow, McGraw-Hill, Inc., New York, 1959.

8. Weir Experiments, Coefficients, and Formulas, R. E. Horton, Water Supply and Irrigation Paper No. 200, Department of the Interior, United States Geological Survey, Washington, D. C., 1907.

9. "Probable Maximum Precipitation Susquehanna River Drainage Above Harrisburg, Pennsylvania," Hydrometeorological Report 40, prepared by H. V. Goodyear and J. T. Riedel, Hydrometeorological Branch Office of Hydrology, U. S. Weather Bureau, U. S. Department of Commerce, Washington, D. C., May 1965.

10. Flood Hydrograph Package (HEC-1) Dam Safety Version, Hydrologic Engineering Center, U. S. Army Corps of Engineers, Davis, California, July 1978.

11. "Simulation of Flow Through Broad Crest Navigation Dams with Radial Gates," R. W. Schmitt, U. S. Army Corps of Engineers, Pittsburgh District.

12. "Hydraulics of Bridge Waterways," BPR, 1970, Discharge Coefficient Based on Criteria for Embankment Shaped Weirs, Figure 24, page 46.

13. Applied Hydraulics in Engineering, Morris, Henry M. and Wiggert, James N., Virginia Polytechnic Institute and State University, 2nd Edition, The Ronald Press Company, New York, 1972.

14. Standard Mathematical Tables, 21st Edition, The Chemical Rubber Company, 1973, page 15.

15. Engineering Field Manual, U. S. Department of Agriculture, Soil Conservation Service, 2nd Edition, Washington, D. C. 1969.

APPENDIX C-1

SUPPLEMENTAL CALCULATIONS

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHUE DAM

BY WJV DATE 4-5-79 PROJ. NO. 79-617-473
CHKD. BY DLB DATE 4-16-79 SHEET NO. 1 OF 15

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

DAM STATISTICS

HEIGHT OF DAM ≈ 34 FT (FIELD MEASURED)

MAXIMUM POOL STORAGE CAPACITY ≈ 270 AC-FT [OBTAINED FROM HEC-1 OUTPUT]
@ TOP OF DAM

NORMAL POOL STORAGE CAPACITY ≈ 215 AC-FT (SEE NOTE 1)

DRAINAGE AREA ≈ 0.38 SQ.MI. (TOTAL)
0.16 SQ MI (LOCAL)
0.22 SQ MI (US OF RAILROAD EMBANKMENT)
[PLANIMETERED OFF USGS 7.5 MINUTE LATROBE, PA QUAD]

NOTE 1: STORAGE VALUE OBTAINED FROM "DAMS, RESERVOIRS, AND NATURAL LAKES", WATER RESOURCES BULLETIN No 5, COMMONWEALTH OF PENNSYLVANIA, DEPARTMENT OF FORESTS AND WATER, HARRISBURG, PA. THE REPORTED VALUE WAS 70 MILLION GALLONS.

DAM CLASSIFICATION

DAM SIZE - SMALL (REF 1, TABLE 1)

HAZARD CLASSIFICATION - HIGH (FIELD OBSERVATION)

REQUIRED SDF - 1/2 PMF TO PMF (REF 1, TABLE 3)

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY WJV DATE 4-5-79 PROJ. NO. 78-617-478

CHKD. BY DLB DATE 4-16-79 SHEET NO. 2 OF 15

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

HYDROGRAPH PARAMETERS

a) FOR SUB-BASIN UPSTREAM OF 75 FT RAILROAD EMBANKMENT (WHICH IS LOCATED JUST US OF THE UPPER DONOHOE RESERVOIR):

LENGTH OF LONGEST WATERCOURSE $(L) \approx 0.78$ MI

$L_{CA} \approx 0.27$ MI (MEASURED ALONG LONGEST WATERCOURSE FROM EMBANKMENT CULVERT INLET TO CENTROID OF SUB-BASIN)

NOTE 2: VALUES OF L AND L_{CA} ARE MEASURED FROM USGS 7.5 MINUTE LATROBE, PA QUAD.

$C_t \approx 1.6$
$C_p \approx 0.45$
} [SUPPLIED BY COE; ZONE 24, OHIO RIVER BASIN]

t_p = SNYDER'S STANDARD LAG $\approx 1.6\,(L \times L_{CA})^{0.3}$

$\therefore t_p \approx 1.6\,[(0.78)(0.27)]^{0.3} \approx 1.0$ HR.

b) FOR LOCAL RESERVOIR SUB-BASIN:

SINCE THE SUB-BASIN CENTROID IS LOCATED WITHIN THE RESERVOIR:

$$^{*}t_p \approx 1.6\,(L')^{0.6}$$

WHERE L' = LENGTH ALONG LONGEST WATERCOURSE FROM THE RESERVOIR BOUNDARY TO THE DRAINAGE DIVIDE ≈ 0.18 MI

$\therefore t_p \approx 1.6\,(0.18)^{0.6} \approx 0.57$ HR ; $C_p \approx 0.45$ (AS ABOVE)

* AS INSTRUCTED BY THE BALTIMORE DISTRICT COE FOR SUCH CASES.

SUBJECT	DAM SAFETY INSPECTION
	UPPER DONOHOE DAM
BY WJV	DATE 4-5-79	PROJ. NO. 78-617-478
CHKD. BY DLB	DATE 4-16-79	SHEET NO. 3 OF 15

GAI CONSULTANTS, INC.

Engineers • Geologists • Planners
Environmental Specialists

RESERVOIR SURFACE AREAS

a) FOR UPSTREAM SUB-BASIN :

ELEVATION (FT)	SURFACE AREA (AC)
≈ 1135	0
1140	2.8
1160	10.1
1180	26.6

NOTE 3 : SURFACE AREAS PLANIMETERED OFF THE 7.5 MINUTE LATROBE, PA QUAD. THE "0" SURFACE AREA ELEVATION WAS ESTIMATED BASED ON THE ASSUMPTION THAT THE ≈ 200 FT EMBANKMENT CULVERT SLOPE WAS EQUAL TO THE 2.5% SLOPE OF THE STREAM IMMEDIATELY UPSTREAM FROM THE CULVERT W/ THE CULVERT OUTLET INVERT @ ABOUT EL 1130 (FIG 2, APPENDIX F)

b) FOR UPPER DONOHOE RESERVOIR :

SURFACE AREA (SA) @ NORMAL POOL EL 1126 ≈ 20.1 ACRES

NOTE 4 : SURFACE AREAS PLANIMETERED OFF FIGURE 2, APPENDIX F. NORMAL POOL ELEVATION ALSO OBTAINED FROM THIS FIGURE. THE NOTES OF FIGURE 2 DO NOT REFER TO THE UPPER DONOHOE DAM.

SA @ EL. 1130 ≈ 24.2 ACRES

SA @ EL. 1135 ≈ 29.4 ACRES

SUBJECT	DAM SAFETY INSPECTION UPPER DONOHOE DAM		
BY WJV	DATE 4-6-79	PROJ. NO. 79-617-479	
CHKD. BY DLB	DATE 4-16-79	SHEET NO. 4 OF 15	

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

RATE OF AREA CHANGE PER FOOT OF RESERVOIR RISE ⇒

$$\Delta A/\Delta H \approx (24.2 - 20.1)\text{ ACRES} / (1130.0 - 1126.0)\text{ FEET} \approx 1.03 \tfrac{AC}{FT}$$

SA @ TOP OF DAM EL. 1123.7 $\approx [(1123.7 - 1126.0) \times 1.03 \tfrac{AC}{FT}] + 20.1$ AC
(LOW TOP OF DAM ELEVATION) $\approx$ 22.9 ACRES
FIELD MEASURED

RESERVOIR ELEVATION @ "O" STORAGE

NORMAL POOL VOLUME $\approx$ 1/3 HA $\approx$ 215 AC-FT (CONIC METHOD)

SA @ NORMAL POOL EL. 1126.0 $\approx$ 20 AC

$$\therefore\ H = \frac{3V}{A} \approx 3(215\text{ AC-FT})/20.1\text{AC} \approx 32.1\text{ FT}$$

ZERO VOLUME ELEVATION $\approx$ 1126.0 FT − 32.1 FT $\approx$ 1093.9 FT

NOTE 5: ACTUAL MINIMUM ELEVATION @ "O" STORAGE IS PROBABLY LESS THAN THE ABOVE VALUE (BASED ON INFORMATION CONTAINED IN PENN DER FILES). HOWEVER, IN ORDER TO COMPUTE A STORAGE-ELEVATION RELATIONSHIP AND STILL MAINTAIN A STORAGE OF 215 AC-FT @ EL 1126.0, THE ABOVE "O" STORAGE ELEVATION MUST BE INPUT INTO THE HEC-1 PROGRAM

STORAGE-ELEVATION RELATIONSHIP

COMPUTED INTERNALLY BY THE HEC-1 PROGRAM FOR BOTH THE UPSTREAM SUB-BASIN AND THE RESERVOIR BASED ON THEIR RESPECTIVE GIVEN SURFACE AREA VS ELEVATION INFORMATION (SEE SUMMARY INPUT / OUTPUT SHEETS).

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHUE DAM

BY WJV DATE 4-5-79 PROJ. NO. 79-617-473

CHKD. BY DLB DATE 4-16-79 SHEET NO. 5 OF 15

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

PMP CALCULATIONS (FOR BOTH THE UPSTREAM AND LOCAL SUB-BASINS)

- APPROXIMATE RAINFALL INDEX = 24 IN (REF 3, FIG 1)
  (CORRESPONDING TO A DURATION OF 24 HR AND AN AREA OF 200 SQ MI LOCATED IN SOUTHWESTERN PENNSYLVANIA)

- DEPTH - AREA - DURATION ZONE #7 (REF 3, FIG 1)

- LOCAL DRAINAGE AREA ≈ 0.16 SQ.MI. AND UPSTREAM DRAINAGE AREA ≈ 0.22 SQ.MI. HOWEVER, THE STORM WILL BE CENTERED OVER THE TOTAL DRAINAGE AREA ABOVE TWIN LAKE Nº1 DAM ≈ 1.89 SQ.MI. (APPENDIX C-1, SHEET 1)
  ⇒ ASSUME THAT DATA CORRESPONDING TO A 10 SQ MI. DA IS REPRESENTATIVE OF THIS BASIN:

DURATION (HR)	PERCENT OF INDEX RAINFALL (%)
6	102.0
12	120.0
24	130.0

NOTE 6: A 24-HR RATHER THAN A 48-HR DURATION IS USED SO THAT A TIME STEP OF 5-MINUTES CAN BE USED IN THE HEC-1 PROGRAM

- HOP BROOK FACTOR (ADJUSTMENT FOR BASIN SHAPE AS WELL AS FOR THE LESSER LIKELIHOOD OF A SEVERE STORM CENTERING OVER A SMALLER BASIN) CORRESPONDING TO A DA ≈ 1.89 SQ.MI. (<10 SQ.MI.) ≈ 0.80 (REF 4, PG 48).

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

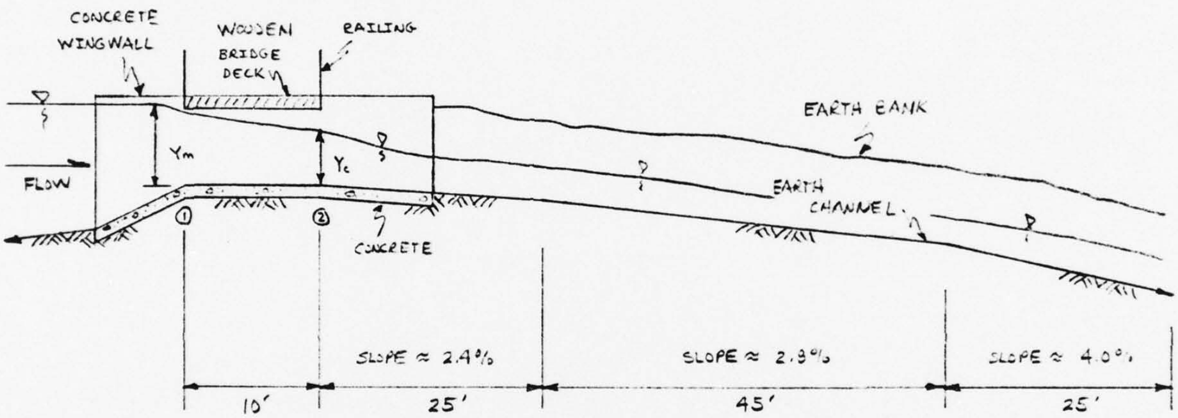
BY WJV DATE 4-6-79 PROJ. NO. 79-617-478

CHKD. BY DLB DATE 4-16-79 SHEET NO. 6 OF 15

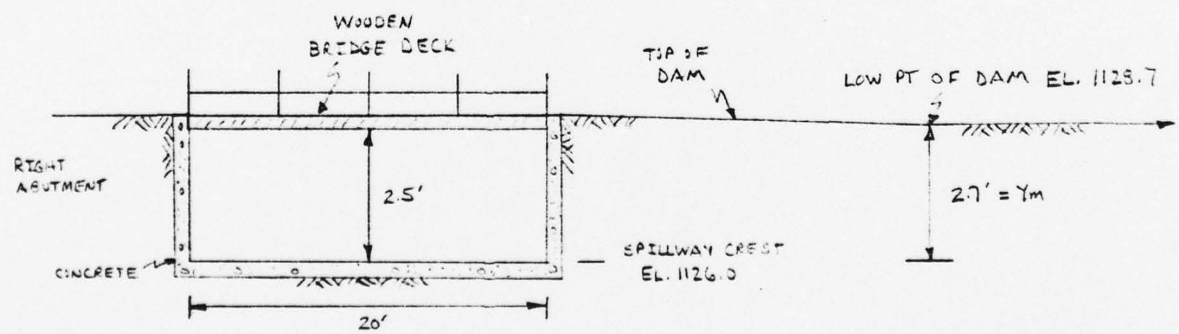
CONSULTANTS, I
Engineers • Geologists • Planners
Environmental Specialists

SPILLWAY CAPACITY

- PROFILE OF SPILLWAY : (NOT TO SCALE)



- SPILLWAY CROSS-SECTION : (NOT TO SCALE)



NOTE 7: SPILLWAY PROFILE SLOPES WERE MEASURED IN THE FIELD AS WERE THE LOW TOP OF DAM ELEVATION AND THE SPILLWAY OPENING DIMENSIONS.

SUBJECT DAM SAFETY INSPECTION
UPPER DONOGHOE DAM

BY WJV DATE 4-3-79 PROJ. NO. 73-617-473

CHKD. BY DLB DATE 4-16-79 SHEET NO. 7 OF 15

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

- ASSUME THAT THE FLOW CONTROL SECTION IS LOCATED @ SECTION ② AS SHOWN ON SHEET 6 W/ Y_c = CRITICAL DEPTH AND Y_m = MAXIMUM RESERVOIR DEPTH ABOVE SPILLWAY CREST PRIOR TO EMBANKMENT OVERTOPPING ≈ 2.7 FT.

ENERGY BALANCE BETWEEN ① AND ②:

$$Y_m + \frac{v_1^2}{2g} + z_1 = Y_c + \frac{v_c^2}{2g} + z_2 + H_L \qquad \text{(REF 7, PG 40)}$$

WHERE v_1 = RESERVOIR VELOCITY ≈ 0 FPS;
z_1 = CHANNEL ELEVATION @ ① IN FT;
v_c = CRITICAL VELOCITY IN FPS;
z_2 = CHANNEL ELEVATION @ ② IN FT;
H_L = HEAD LOSS BETWEEN ① AND ② ≈ 0 FT

SINCE $z_1 - z_2 \approx 0$ (SECTIONS ① AND ② ARE CLOSE ENOUGH TOGETHER SUCH THAT Δ ELEVATION ≈ 0)

$$Y_m = 2.7\text{ FT} = Y_c + \frac{v_c^2}{2g}$$

- SINCE THE CRITICAL SECTION IS RECTANGULAR IN SHAPE,

$$\frac{v_c^2}{2g} = \frac{Y_c}{2} \qquad \text{(REF 13, PG 143)}$$

$$\therefore \quad 2.7\text{ FT} = Y_c + \frac{v_c^2}{2g} = Y_c + \frac{Y_c}{2} = \frac{3}{2} Y_c$$

$$Y_c \approx 1.8\text{ FT}$$

- SINCE $Y_c \approx 1.8$ FT ⇒ $A_c = 20\,Y_c \approx (20)(1.8) \approx 36\text{ FT}^2$
$v_c \approx \sqrt{2g\,(Y_c/2)} \approx \sqrt{g\,Y_c} \approx \sqrt{(32.2\text{ FT/SEC}^2)(1.8)}$
≈ 7.6 FPS

∴ CAPACITY OF SPILLWAY = $Q = A_c v_c \approx (36\text{ FT}^2)(7.6\text{ FPS}) \approx 274$ CFS
SAY 270 CFS

SUBJECT DAM SAFETY INSPECTION

UPPER DONOHOE DAM

BY WJV DATE 5-4-79 PROJ. NO. 79-617-497

CHKD. BY DLB DATE 5-4-79 SHEET NO. 8 OF 15

GAI CONSULTANTS, INC.

Engineers • Geologists • Planners
Environmental Specialists

- CHECK TO SEE IF CRITICAL DEPTH DOES CONTROL @ ② :

CHANNEL SLOPE DS OF ② ≈ 2.4% (SHEET 6)

CRITICAL SLOPE IS DEFINED BY MANNING'S EQ :

$$S_c \approx \left(\frac{n\, v_c}{1.49\, R^{2/3}} \right)^2$$ (REF 13, PG 143)

WHERE n = CHANNEL ROUGHNESS COEFFICIENT ≈ 0.025 (FIELD ESTIMATE FOR EARTH CHANNEL); $v_c \approx 7.6$ FPS (SHEET 7), AND R = HYDRAULIC RADIUS = $\frac{\text{FLOW AREA}}{\text{WETTED PERIMETER}}$ $\approx A_c/P \approx 36\,FT^2/(20+1.8+1.8) \approx 1.53$ FT

$$\therefore\ S_c \approx \left[(0.025)(7.6) / (1.49)(1.53)^{2/3} \right]^2 \approx 0.92\% < 2.4\%$$

⇒ CRITICAL DEPTH CONTROLS @ SECTION ②

NOTE 8: IF THE EMBANKMENT WAS ACTUALLY LEVEL AND AT DESIGN EL 1130 ⇒ $Y_m \approx 4$ FT ⇒ $Y_c \approx 2.67$ FT ⇒ $v_c \approx 9.3$ FPS. THEREFORE, $A_c \approx 20(2.67) \approx 53.4\ FT^2$ ⇒ $Q = A_c v_c \approx (53.4\ FT^2)(9.3\ FPS)$ $Q \approx 500$ CFS (ASSUMING THAT THE SPILLWAY BRIDGE WILL BE WASHED AWAY). IF THE SPILLWAY CAPACITY ACTUALLY WAS ≈ 500 CFS ⇒ THE FACILITY COULD ACCOMMODATE A FLOOD IN EXCESS OF 70% OF THE PMF (SUMMARY INPUT/OUTPUT SHEETS, SHEET H).

SPILLWAY RATING CURVE

COMPUTED INTERNALLY BY HEC-1 VIA THE TRAPEZOIDAL RATING CURVE ROUTINE, BASED ON THE SPILLWAY GEOMETRY AS PRESENTED ON SHEET 6. THE TRAPEZOIDAL ROUTINE CALCULATES CRITICAL CONTROL DISCHARGES IN A WAY SIMILAR TO THAT OUTLINED ON SHEET 7 (SEE SUMMARY INPUT / OUTPUT SHEETS).

SUBJECT	DAM SAFETY INSPECTION				
	UPPER DONOHOE DAM				
BY	WJV	DATE	4-9-79	PROJ. NO.	78-617-479
CHKD. BY	DLB	DATE	4-16-79	SHEET NO.	9 OF 15

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

RAILROAD EMBANKMENT CULVERT RATING CURVE

- CULVERT INLET ≈ 2.5 FT (DEPTH) × 3.0 FT (WIDTH) RECTANGULAR MASONRY OPENING (FIELD MEASURED)

- CULVERT OUTLET => REPORTED TO BE A 6 FT DIAMETER PIPE. HOWEVER, ON THE DAY OF INSPECTION THE OUTLET WAS SUBMERGED WITH WATER, DUE TO THE ACCUMULATION OF ABOUT 5± FT OF SEDIMENT IN THE CULVERT AND EXIT CHANNEL. THIS ACCUMULATION OF SEDIMENT IS REPORTED TO BE A COMMON OCCURRENCE W/ DREDGING DONE INFREQUENTLY.

SINCE THE ACTUAL CULVERT OUTLET OPENING SIZE IS NOT KNOWN, AND DUE TO THE LARGE SEDIMENT ACCUMULATIONS WHICH CONSTANTLY CLOG MOST OF THE OUTLET OPENING, THE PERFORMANCE OF THE OUTLET WILL BE ASSUMED TO BE REPRESENTED BY THE PERFORMANCE OF A 2.5 FT × 3.0 FT RECTANGULAR OPENING.

- CULVERT DISCHARGES ARE CONTROLLED BY EITHER THE INLET OR THE OUTLET OF THE CULVERT; DEPENDING ON SUCH FACTORS AS CROSS-SECTIONAL AREA, LENGTH, ROUGHNESS, SLOPE, AND ENTRANCE CONDITIONS OF THE CULVERT BARREL, AS WELL AS HEADWATER AND TAILWATER LEVELS.

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY WJV DATE 4-9-79 PROJ. NO. 78-617-473

CHKD. BY DLB DATE 4-16-79 SHEET NO. 10 OF 15

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

- *INLET CONTROL DISCHARGES ARE INDEPENDENT OF TAILWATER DEPTH, AND ARE CONTROLLED BY HEADWATER LEVEL AND ENTRANCE GEOMETRY. FOR H/D (HEADWATER DEPTH TO CULVERT DEPTH RATIO) < 1.2, THE DISCHARGE EQUATION IS:

$$Q = \frac{2}{3} C_B B H \sqrt{\frac{2}{3} g H} \quad \text{(CONSTRICTED FLOW)}$$

WHERE Q = DISCHARGE IN CFS; C_B = END CONSTRICTION COEFFICIENT ≈ 0.9 (SQUARE EDGED ENTRANCE), B = WIDTH OF CULVERT = 3.0 FT, H = HEADWATER DEPTH ABOVE INLET INVERT ELEVATION OF 1135.0 FT, AND g = 32.2 FT/SEC².
↳ (ESTIMATED, SEE SHEET 3)

FOR H/D > 1.2:

$$Q = C_h B D \sqrt{2g(H - C_h D)}$$

WHERE Q, B, g, AND H ARE AS BEFORE, D = DEPTH OF CULVERT = 2.5 FT, AND C_h = CONTRACTION COEFFICIENT = 0.6 (SQUARE-EDGED ENTRANCE).

* INFORMATION OBTAINED FROM: OPEN CHANNEL FLOW BY F.M. HENDERSON, MACMILLAN PUBLISHING CO., INC., NEW YORK, NEW YORK. 1966 (PG 263)

- INLET CONTROL FLOWS:

ELEVATION (FT)	H (FT)	H/D (FT/FT)	Q (CFS)	ELEVATION (FT)	H (FT)	H/D (FT)	Q (CFS)
1135.0	0	0	0	1142.0	7	2.8	85
1136.0	1	0.4	10	1143.0	8	3.2	90
1137.0	2	0.8	20	1144.0	9	3.6	100
1138.0	3	1.2	40	1145.0	10	4.0	105
1139.0	4	1.6	60	1146.0	11	4.4	110
1140.0	5	2.0	70	1147.0	12	4.8	115
1141.0	6	2.4	80	1148.0	13	5.2	120

SUBJECT	DAM SAFETY INSPECTION
	UPPER DONOHOE DAM
BY WJV	DATE 4-9-79	PROJ. NO. 78-617-473
CHKD. BY DLB	DATE 4-16-79	SHEET NO. 11 OF 15

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

- **OUTLET CONTROL DISCHARGES ARE ESPECIALLY DEPENDENT ON TAILWATER LEVEL, ALONG WITH ALL OTHER CHARACTERISTICS OF THE CULVERT BARREL. OUTLET CONTROL CAN OCCUR IF H > 0.75 D, WITH DISCHARGE DEFINED BY ITS RELATIONSHIP TO HW IN THE EQUATION BELOW:

$$HW = \left[1 + K_e + \frac{29 n^2 L}{R^{1.33}}\right] \frac{Q^2}{2gA^2} + TW - LS_o$$

WHERE HW = WATER SURFACE ELEVATION @ INLET IN FT; K_e = ENTRANCE LOSS COEFFICIENT ≈ 0.4 (WINGWALLS @ 30°-75° TO CULVERT, SEE REF BELOW); $n \approx 0.020$ (FIELD ESTIMATE); $A = 7.5\ FT^2$; $R = \frac{FLOW\ AREA}{WETTED\ PERIMETER} = 7.5/[3+3+2.5+2.5] \approx 0.68$ FT; L = LENGTH OF CULVERT ≈ 200 FT (ESTIMATED); S_o = SLOPE OF CULVERT ≈ 0.025 (SHEET 3); Q = CULVERT DISCHARGE IN CFS; TW = TAILWATER ELEVATION = ELEVATION OF OUTLET INVERT (≈ 1130.0 FT) + DEPTH OF CULVERT (2.5 FT) FOR Q UP TO FLOW AT WHICH OUTLET CONTROL OVERTAKES INLET CONTROL, THEN ASSUME THAT TW INCREASES ABOVE THIS ELEVATION BY 0.1 FT FOR EVERY 10 CFS OR SO INCREASE IN FLOW.

** INFORMATION OBTAINED FROM: "HYDRAULIC CHARTS FOR THE SELECTION OF HIGHWAY CULVERTS", HEC No 5, BUREAU OF PUBLIC ROADS.

- OUTLET CONTROL FLOWS:

Q (CFS)	TW (FT)	LS_o (FT)	HW (FT)	Q (CFS)	TW (FT)	LS_o (FT)	HW (FT)
60	1132.5	5	1132.7	120	1132.6	5	1148.6
70	1132.5	5	1134.6	130	1132.7	5	1152.3
80	1132.5	5	1136.8	140	1132.8	5	115[illegible].3
90	1132.5	5	1139.3	150	1132.9	5	1160.7
100	1132.5	5	1142.1	160	1133.0	5	1165.3
110	1132.5	5	1145.1	170	1133.1	5	1170.2
[illegible]	[illegible]	5	[illegible]				

SUBJECT	DAM SAFETY INSPECTION UPPER DONOHOE DAM					
BY	WJV	DATE	4-9-79	PROJ. NO.	78-617-478	
CHKD. BY	DLB	DATE	4-16-79	SHEET NO.	12	OF 15

GAI CONSULTANTS, INC.

Engineers • Geologists • Planners
Environmental Specialists

– TOTAL CULVERT RATING CURVE:

HEADWATER ELEVATION (FT)	① Q_{INLET} (CFS)	② Q_{OUTLET} (CFS)	Q (CFS)
1135.0	0	–	0
1136.0	10	76	10
1137.0	20	81	20
1138.0	40	85	40
1139.0	60	89	60
1140.0	70	93	70
1141.0	80	96	80
1142.0	85	100	85
1143.0	90	103	90
1144.0	100	106	100
1145.0	105	110	105
1146.0	110	113	110
1147.0	115	116	115
1147.4	117	117	117
1148.6		120	120
1152.3		130	130
1156.3		140	140
1160.7		150	150
1165.3		160	160
1170.2		170	170

① FROM SHEET 10
② FROM SHEET 11

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY WJY DATE 4-9-79 PROJ. NO. 78-617-479
CHKD. BY DLB DATE 4-16-79 SHEET NO. 13 OF 15

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

DAM EMBANKMENT RATING CURVE

- ALTHOUGH THE EMBANKMENT CREST IS LINED WITH ≈ 0.6 FT WOODEN RAILROAD TIES, ANY ADDITIONAL STORAGE WHICH THEY MIGHT PROVIDE WILL BE NEGLECTED SINCE THEY ARE NOT CONSIDERED TO BE PERMANENT STRUCTURES ⇒ ASSUME THE WOODEN TIES ARE REMOVED.

- FLOWS OVER THE EMBANKMENT WILL BE COMPUTED INTERNALLY BY HEC-1 VIA THE ASSUMPTION THAT CRITICAL DEPTH OCCURS ON THE CREST W/ THE CREST PROFILE REPRESENTED BY A SERIES OF TRAPEZOIDS. (SEE SUMMARY INPUT/OUTPUT SHEETS FOR RATING INFORMATION).

- INPUT INFORMATION: (BASED ON FIELD MEASUREMENTS)

RESERVOIR ELEVATION (FT)	DEPTH OF WATER ABOVE CREST (FT)	LENGTH OF CREST INUNDATED (FT)	
1128.7	0	0	
1128.8	0.1	75	
1128.9	0.2	175	
1129.0	0.3	225	
1129.1	0.4	325	
1129.2	0.5	375	
1129.3	0.6	420	
1129.5	0.8	500	
1129.6	0.9	520	
1129.7	1.0	521	ASSUME 20% SLOPES TO THE RIGHT AND LEFT OF THE EMBANKMENT
1130.2	1.5	526	
1130.7	2.0	531	

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY WJV DATE 5-4-79 PROJ. NO. 78-617-437

CHKD. BY DLB DATE 5-4-79 SHEET NO. 14 OF 15

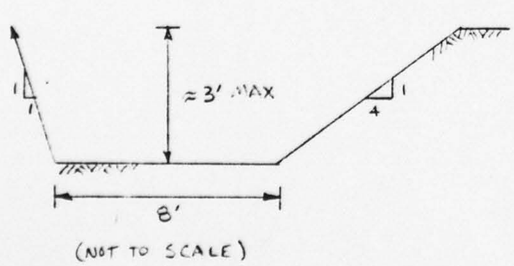
CONSULTANTS, I
Engineers • Geologists • Planners
Environmental Specialists

ESTIMATE OF ACTUAL SPILLWAY CAPACITY

- ALTHOUGH THE HYDROLOGIC/HYDRAULIC EVALUATION OF THIS FACILITY WILL BE PERFORMED ASSUMING THAT THE SPILLWAY CHUTE CHANNEL IS IN IDEAL CONDITION (SEE SECTION 5.3), SOME ESTIMATE OF THE ACTUAL PRESENT SPILLWAY SYSTEM CONSEQUENCES AND CAPACITY SHOULD BE MADE.

- DUE TO THE PRESENCE OF THE BREACH IN THE LEFT CHUTE CHANNEL WALL (CAUSED BY THE PLACEMENT OF A TEMPORARY EARTH AND ROCK ROAD ACROSS THE CHUTE CHANNEL @ ABOUT 35 FT DS FROM THE SPILLWAY CREST), SOME SPILLWAY DISCHARGE WILL FLOW TOWARD THE TOE WHEN THE CHANNEL DEPTH EXCEEDS ABOUT 1.5 FT OR SO.

- THE ACTUAL PRESENT CRITICAL FLOW CONTROL SECTION IS LOCATED ABOUT 70 FT DS FROM THE SPILLWAY (SEE SKETCH ON SHEET 6). THE APPROXIMATE CROSS-SECTION DIMENSIONS ARE GIVEN IN THE SKETCH BELOW. ASSUMING UNIFORM FLOW ABOVE THIS SECTION (REF 7, PG. 5), THE MAXIMUM DEPTH OF FLOW UPSTREAM FROM THE SECTION WILL BE ≈ 2.7 FT (CORRESPONDING TO THE MAXIMUM SPILLWAY DEPTH PRIOR TO OVERTOPPING W/O THE CRITICAL CONTROL ON THE CREST). ALSO, IGNORING THE POSSIBLE FLOW THROUGH THE LEFT CHUTE CHANNEL WALL BREACH THE SPILLWAY CAPACITY CAN BE FOUND



(NOT TO SCALE)

FROM: $Y_m = Y_c + v_c^2/2g \Rightarrow 2.7 = Y_c + v_c^2/2g$. SINCE $A_c \approx 8Y_c + 2.5Y_c^2$ AND B_c = (TOP WIDTH) = $8 + 5Y_c$ (FROM GEOMETRY); $Q = A_c v_c$; AND $Q^2 B_c = g A_c^3$ (REF 13, PG 41) $\Rightarrow$

$$Q \approx \sqrt{g[8Y_c + 2.5Y_c^2]^3/[8+5Y_c]}, \text{ AND}$$

(ASSUMING APPROACH VELOCITY HEAD AND CHANNEL LOSSES ARE NEGLIGIBLE)

$$2.7 = Y_c + \frac{Q^2}{2g(8Y_c + 2.5Y_c^2)^2}$$

SUBJECT DAM SAFETY INSPECTION

UPPER DONOHOE DAM

BY WJV DATE 5-4-79 PROJ. NO. 78-617-487

CHKD. BY DLB DATE 5-4-79 SHEET NO. 15 OF 15

GAI CONSULTANTS, INC.

Engineers • Geologists • Planners
Environmental Specialists

THEREFORE, BY TRIAL AND ERROR $\Rightarrow$ $Y_c \approx 2.0$ FT

$Q \approx 180$ CFS

THUS, THE CAPACITY OF THE ACTUAL PRESENT SPILLWAY SYSTEM IS ABOUT 2/3 OF THAT COMPUTED FOR THE PROPOSED REHABILITATED SPILLWAY SYSTEM (SHEET 7). HOWEVER, DUE TO THE BREACH IN THE LEFT SPILLWAY CHANNEL WALL, THE ABOVE COMPUTED CAPACITY PRIOR TO OVERTOPPING WILL ACTUALLY BE SOMEWHAT MORE.

APPENDIX D

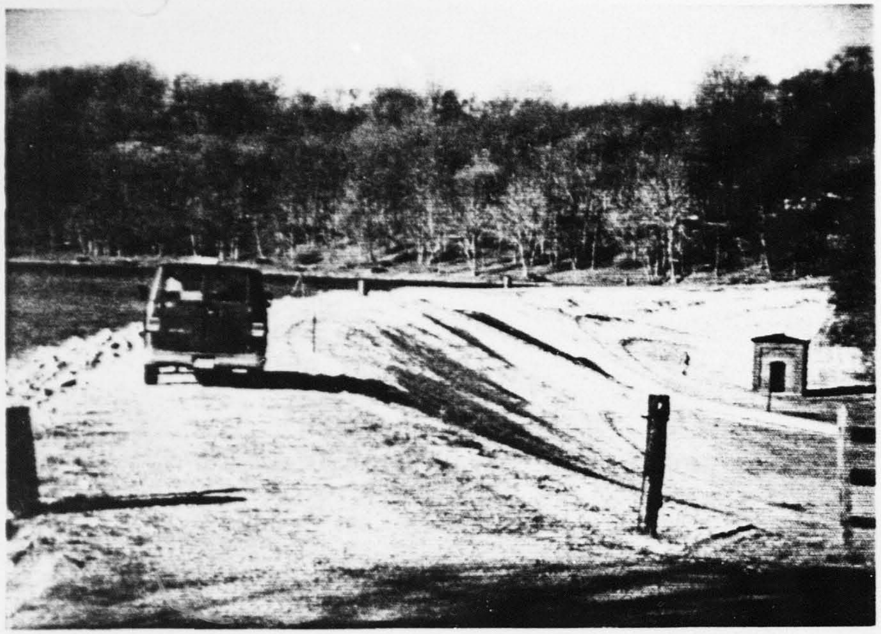
PHOTOGRAPHS

PHOTOGRAPH 1 View looking west along the crest and downstream slope of the embankment.

PHOTOGRAPH 2 View looking west along the riprapped upstream slope of the embankment.

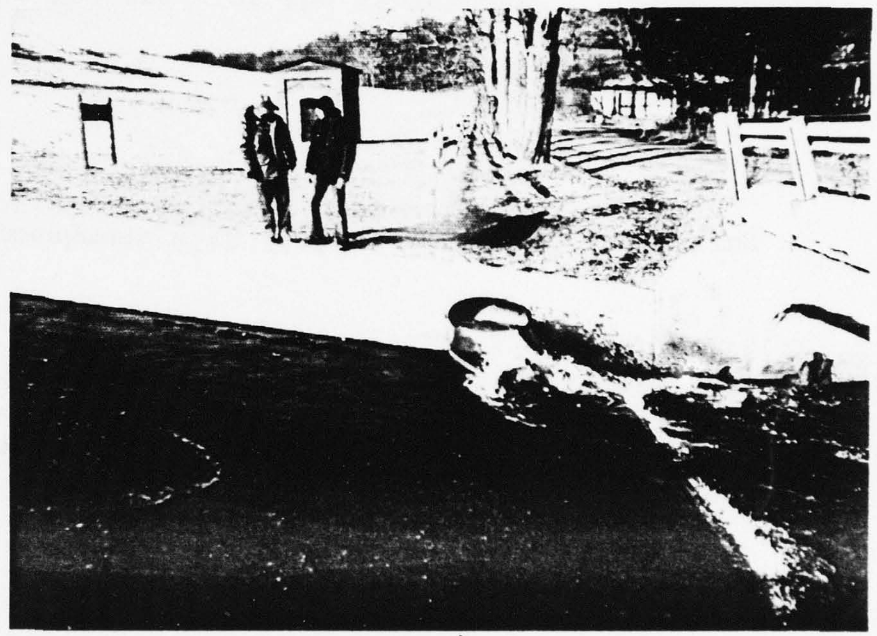
PHOTOGRAPH 3 View looking west along the downstream toe of the embankment. Inspection team personnel are within areas of noticeable saturation.

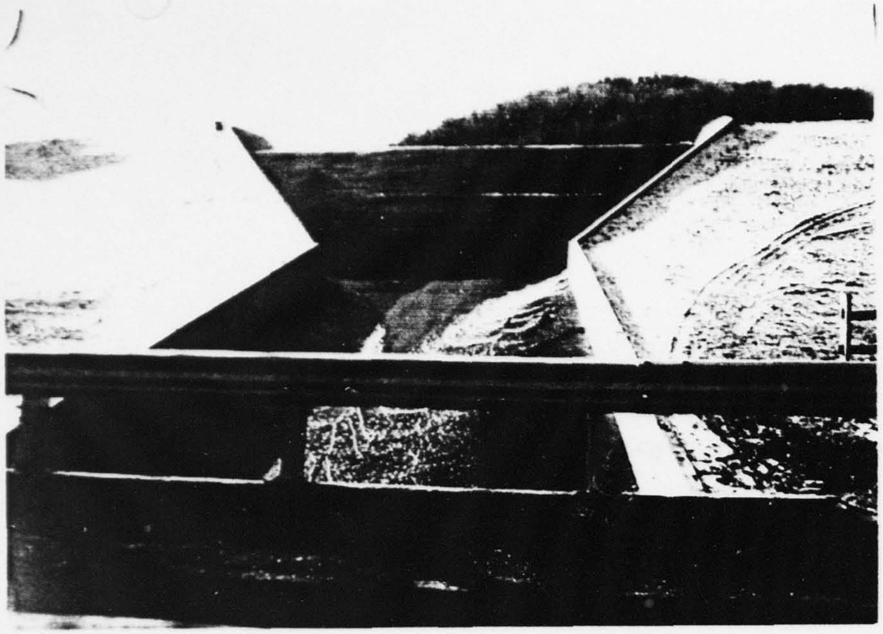
PHOTOGRAPH 4 View of outlet works discharging near toe of spillway.


1


2

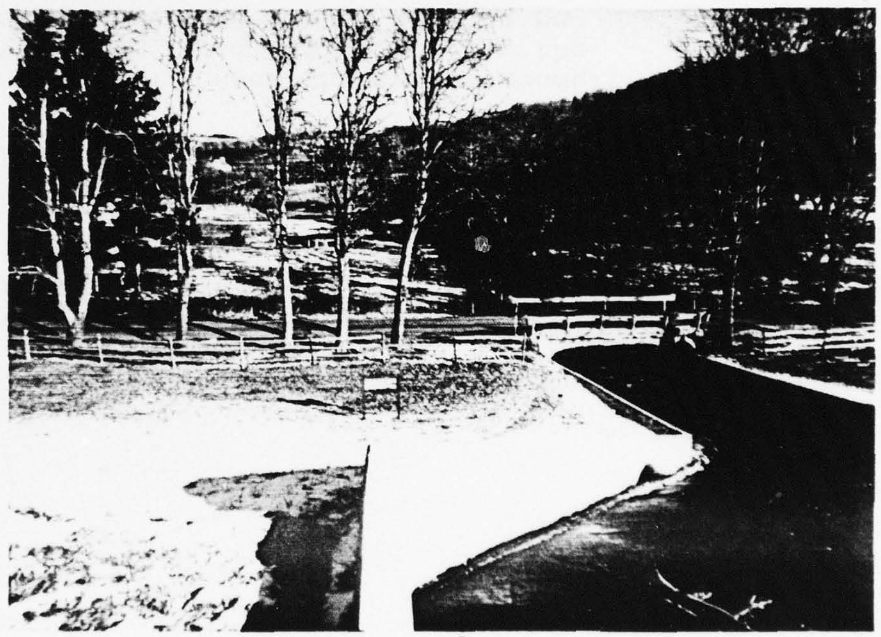

3


4


5


6


7


8

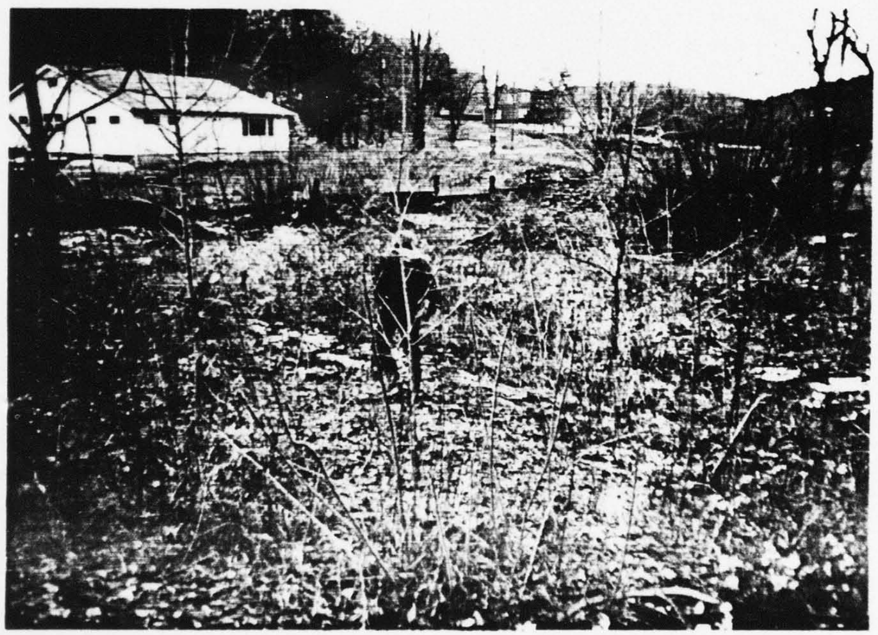
PHOTOGRAPH 9 View from the right abutment showing reservoir area and Upper Donohoe Dam in the upper left background.

PHOTOGRAPH 10 View of the access manhole which houses the outlet conduit control valve. The valve wheel is immediately below the lower rim of the manhole on an extended stem and can be operated without entering the manhole.

PHOTOGRAPH 11 View, looking south, of a permanent dwelling approximately 2,500 feet downstream of the dam.

9

10



11

APPENDIX E

GEOLOGY

Geology

Twin Lakes No. 1 Dam is located in the Pittsburgh Plateaus Section of the Appalachian Plateaus Physiographic Province. The Pittsburgh Plateaus Section is characterized by flat lying to very gently folded sedimentary rock strata of Pennsylvanian age. Major structural axes strike from southwest to northeast with flanking strata dipping northwest and southeast. The amplitude of folding in this section is quite low; consequently, surface expression of the anticlinal axes is not evident. More specifically, the site lies on the western flank of the Fayette anticline. Bedrock at the site dips to the northwest at approximately 300 feet per mile.

The dam and reservoir are developed wholly on sedimentary rock strata of the Conemaugh Group of Pennsylvanian age. Based on published data, the bedrock underlying the foundation of the dam contains those members of the Conemaugh Group which generally lie approximately 250 to 280 feet below the base of the Pittsburgh Coal seam. The generalized stratigraphic column for this area indicates the Ames Limestone, a well known marker bed, should lie at approximate elevation 1075.

In 1973, Geo-Mechanics, Inc., conducted a subsurface investigation of the existing embankment to evaluate the structure and develop rehabilitation design parameters. A total of 13 test borings were drilled on the existing

embankment. Nine of these borings penetrated the bedrock underlying the dam. The following excerpt is taken from Geo-Mechanics' "Rehabilitation Investigation."[1]

"The depth of bedrock below the natural ground surface varies from about 10 feet at each abutment to about 20 feet near the middle of the dam. The elevation of top of rock slopes from about 1070 near the abutments to about 1050 near the middle of the dam. A study of the bedrock strata indicates that the bedrock is dipping strongly from the right abutment towards the left abutment which is in agreement with the information obtained from the published data. There is a drop of almost 10 feet in 200 feet. As a result, the top of bedrock varies from silty shale to shaley limestone to shaley siltstone. There are two distinct strata of limestone which are encountered near the top of rock; one at Station 6+00 and the other at Station 2+50. Immediately above the bedrock, a 5- to 8-foot thick zone consisting of very badly broken rock is found which is classified as rock fragments and is residual in nature, that is, formed by the in-place weathering of the underlying bedrock and retaining most of the characteristics of the parent rocks.

The foundation soils at the dam site consists of residual soils along the abutments and both alluvial and residual soils in the floodplain. The thickness of soil zone varies generally from about 10 feet to 20 feet. The alluvial soils have been transported and deposited by the stream action."

The limestone encountered in several of the core borings is probably the Ames Limestone and a typical profile is shown in Figure 6.

---

1. "Rehabilitation Investigation, Lower Dam, Twin Lakes Park, Greensburg, Pennsylvania," prepared by Geo-Mechanics, Inc., for the Westmoreland County, Department of Parks and Recreation, Greensburg, Pennsylvania, 1973.
2. "Geologic Atlas of the United States, Latrobe Folio, Pennsylvania," U. S. Geological Survey, No. 110, 1904.
3. "Generalized Stratigraphic Section for the Greater Pittsburgh Region," B. J. O'Neill, Jr., Topographic and Geologic Survey, M67, Plate 3, Harrisburg, Pennsyl vania, 1974.

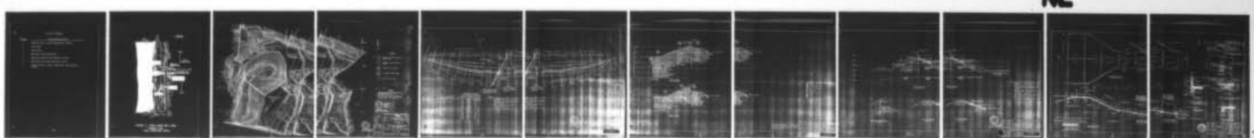
APPENDIX F

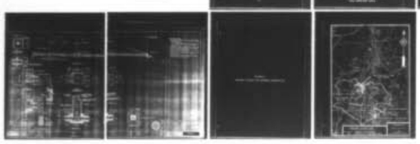
FIGURES

AD-A070 585 GAI CONSULTANTS INC MONROEVILLE PA F/G 13/2
NATIONAL DAM INSPECTION PROGRAM. TWIN LAKES NUMBER 1 DAM (NDS1.--ETC(U)
APR 79 DACW31-79-C-0013

UNCLASSIFIED NL

2 OF 2
AD
A070585





END
DATE
FILMED
8 -79
DDC

LIST OF FIGURES

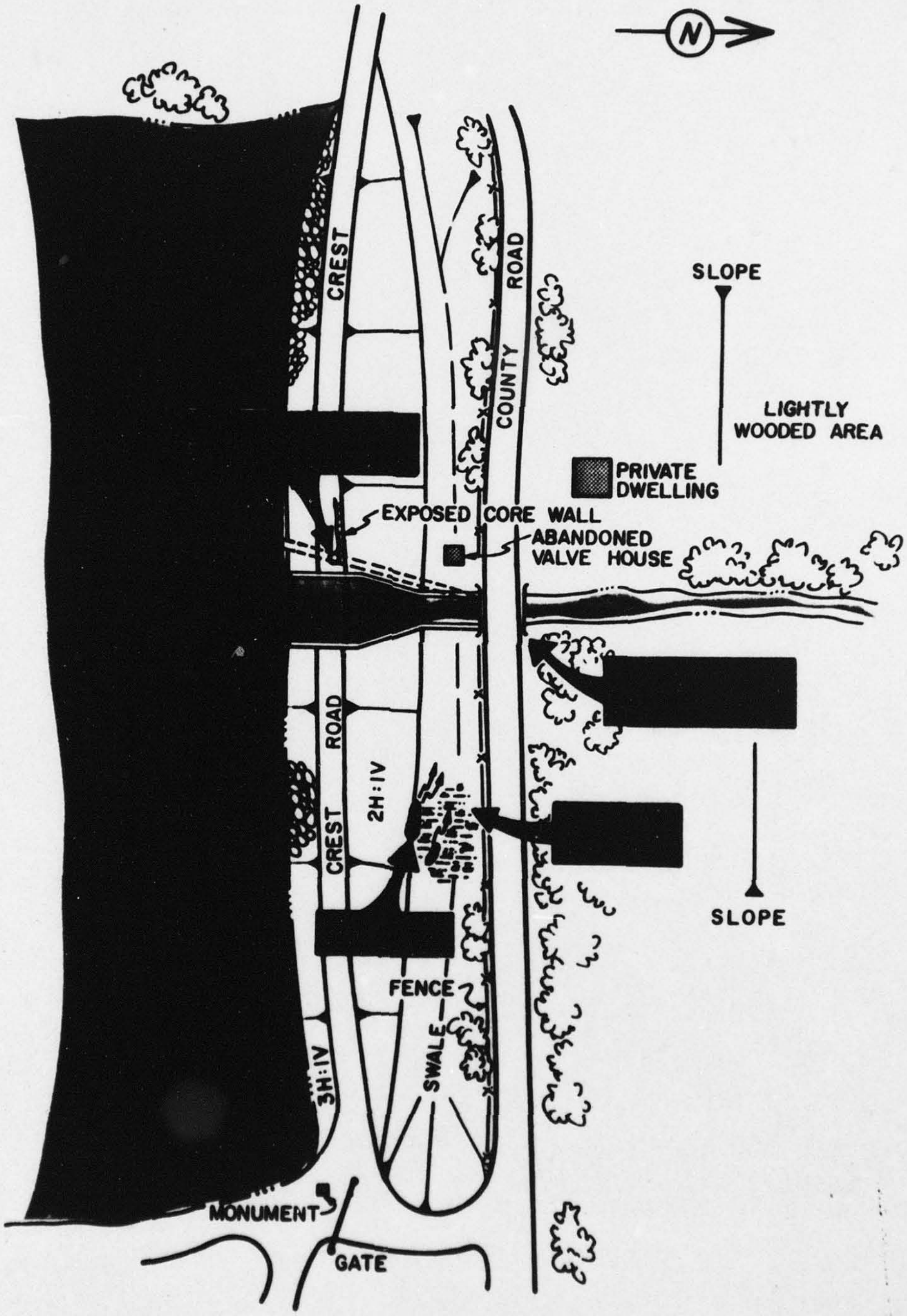
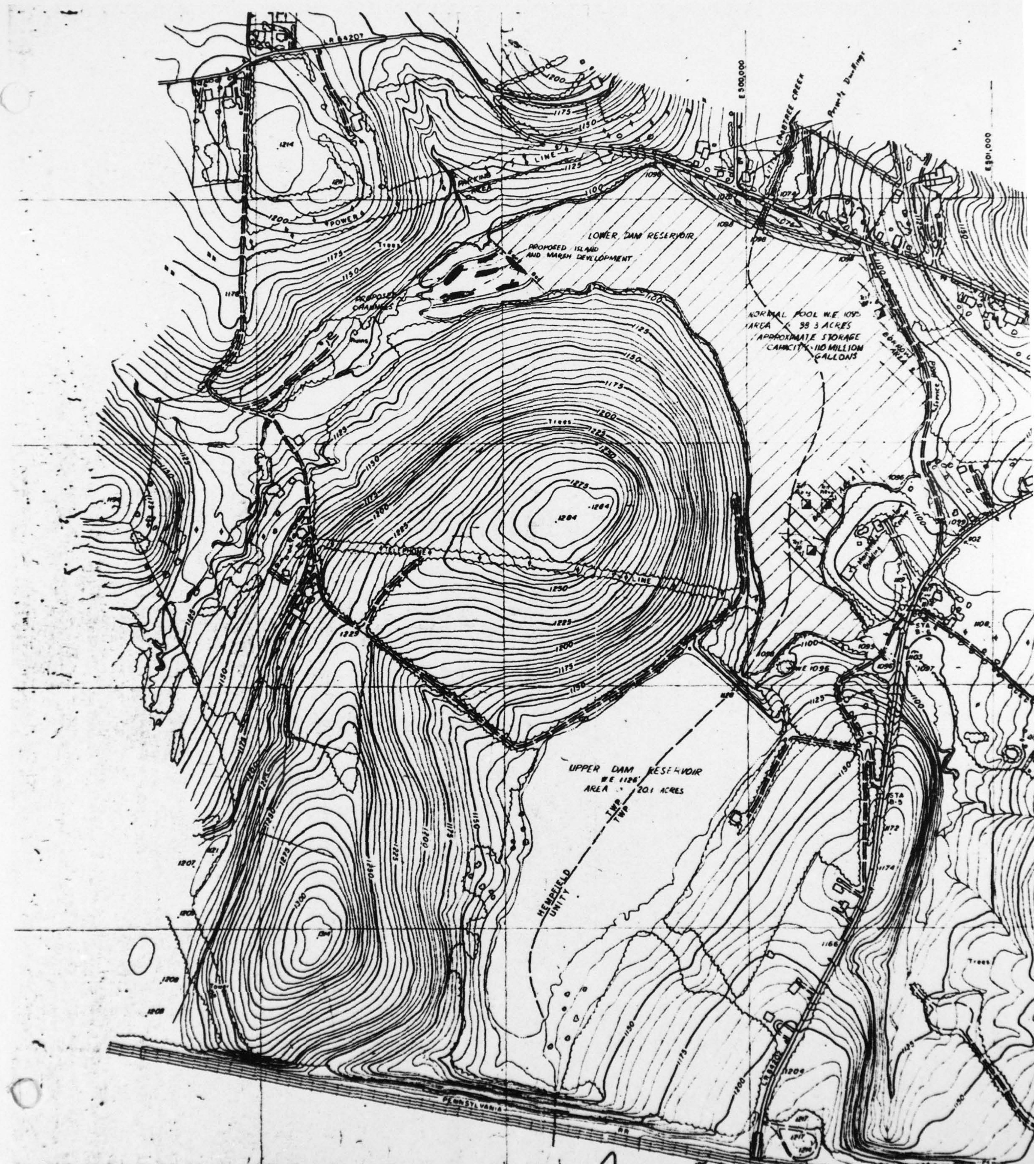


FIGURE 1 - TWIN LAKES NO. 1 DAM
GENERAL PLAN
FIELD INSPECTION NOTES


LOWER DAM RESERVOIR
PROPOSED ISLAND AND MARSH DEVELOPMENT
PROPOSED CHANNELS
NORMAL POOL W.E. 1093
AREA 39.3 ACRES
APPROXIMATE STORAGE CAPACITY 110 MILLION GALLONS
UPPER DAM RESERVOIR
W.E. 1126'
AREA 20.1 ACRES
HEMPFIELD TWP
UNITY TWP
CRABTREE CREEK
Private Dwellings
PARKING AREA
POWER LINE
E 300,000
E 301,000
PENNSYLVANIA RR

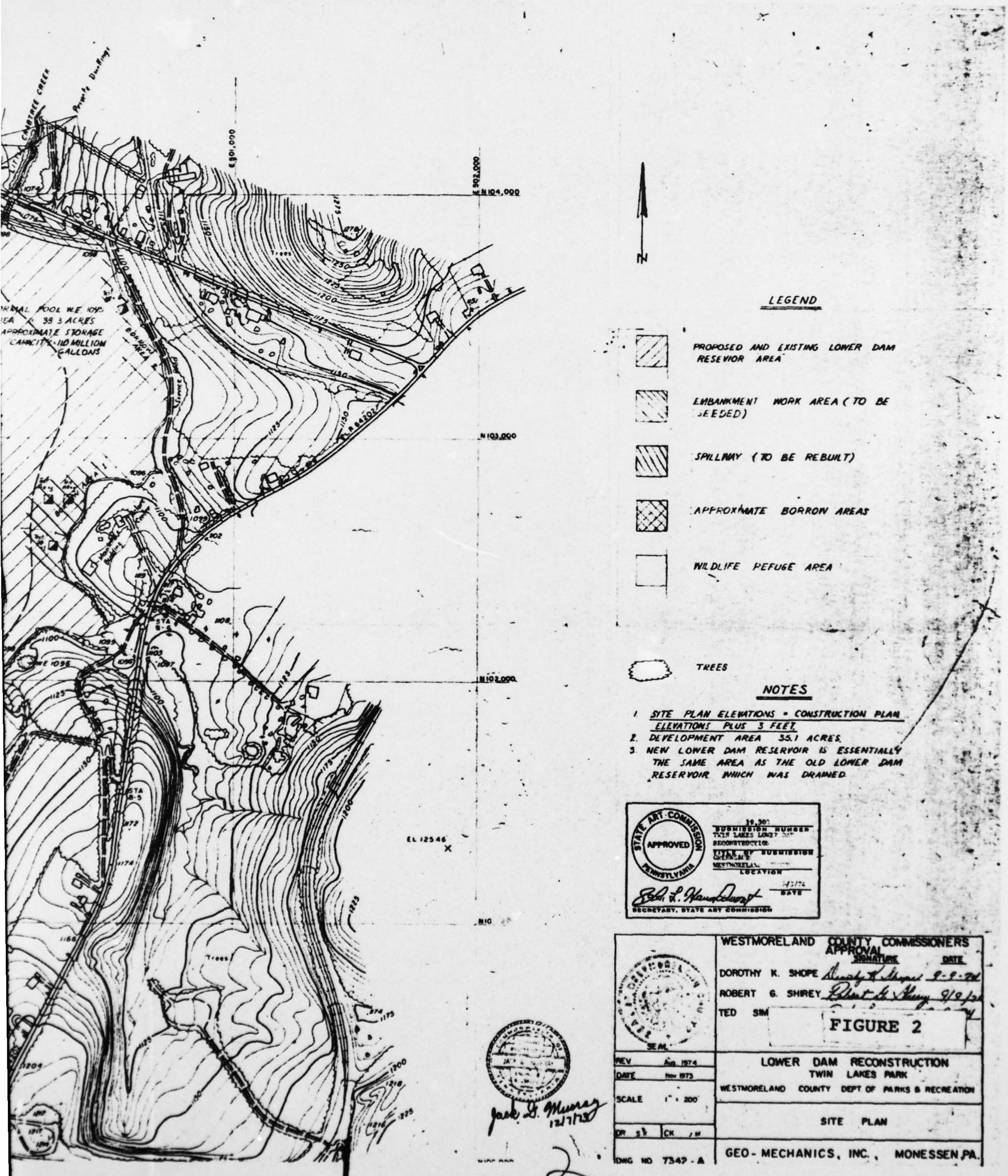
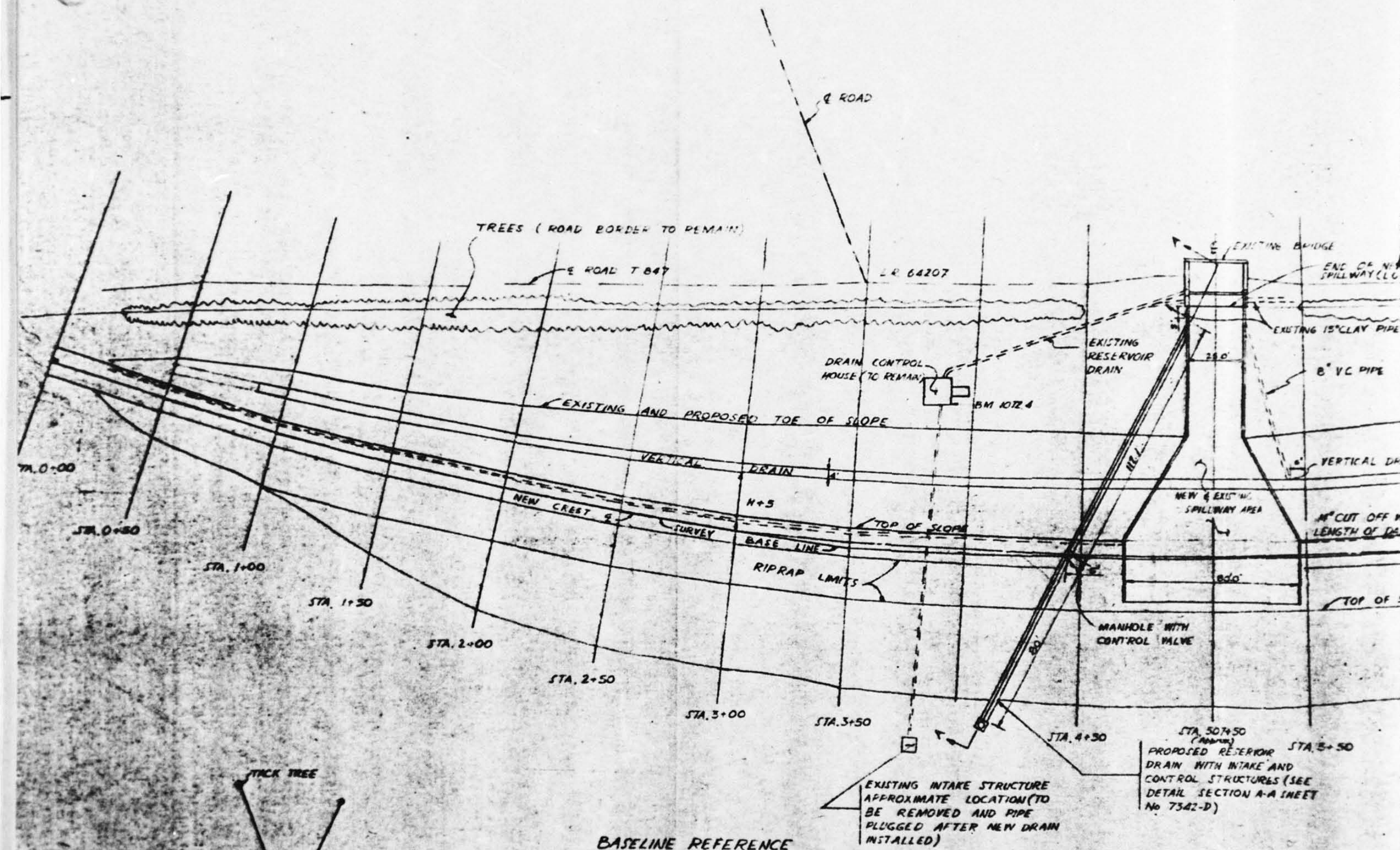
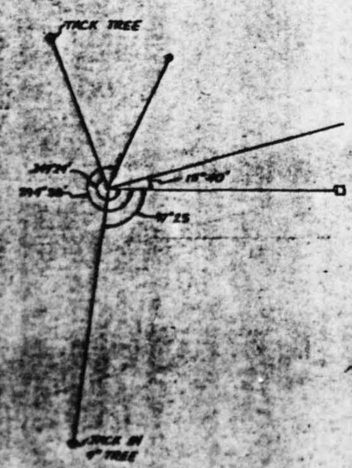

CRABTREE CREEK
Proposed Dwellings
E 901,000
E 902,000
N 104,000
N 103,000
N 102,000
NORMAL POOL W.E. 1095
AREA 39.3 ACRES
APPROXIMATE STORAGE
CAPACITY 110 MILLION GALLONS
L.R. 64202
EL 1254.6
LEGEND
PROPOSED AND EXISTING LOWER DAM RESEVIOR AREA
EMBANKMENT WORK AREA (TO BE SEEDED)
SPILLWAY (TO BE REBUILT)
APPROXIMATE BORROW AREAS
WILDLIFE REFUGE AREA
TREES
NOTES
1. SITE PLAN ELEVATIONS = CONSTRUCTION PLAN ELEVATIONS PLUS 3 FEET.
2. DEVELOPMENT AREA 35.1 ACRES.
3. NEW LOWER DAM RESERVOIR IS ESSENTIALLY THE SAME AREA AS THE OLD LOWER DAM RESERVOIR WHICH WAS DRAINED.
STATE ART COMMISSION
APPROVED
PENNSYLVANIA
SUBMISSION NUMBER
TWIN LAKES LOWER DAM RECONSTRUCTION
WESTMORELAND
LOCATION
DATE
SECRETARY, STATE ART COMMISSION
WESTMORELAND COUNTY COMMISSIONERS APPROVAL
SIGNATURE
DATE
DOROTHY K. SHOPE
9-9-74
ROBERT G. SHIREY
9/9/74
TED SIM
SEAL
REV Aug 1974
DATE Nov 1973
SCALE 1" = 200'
LOWER DAM RECONSTRUCTION
TWIN LAKES PARK
WESTMORELAND COUNTY DEPT OF PARKS & RECREATION
SITE PLAN
DWG NO 7347-A
GEO-MECHANICS, INC., MONESSEN, PA.
Jack G. Murray 12/7/73
2

FIGURE 2





REFERENCE FOR
STA 0+00
NO SCALE

BASELINE REFERENCE

DEFLECTION ANGLES ARE MEASURED FROM STA. 0+00 SIGHTING ON STA. 0+50 AND TURNED TO THE LEFT

STA. No.	DEFLECTION ANGLE	APPROXIMATE OFFSET TO NEW ℄ OF CREST
0+50	0°00'	0.0'
1+00	0°46'	0.0'
1+50	1°46'30"	0.0'
2+00	2°45'	0.0'
2+50	3°45'	0.0'
3+00	4°40'30"	0.0'
3+50	5°37'30"	-2.7'
4+00	6°42'30"	-2.2'
4+50	7°41'30"	-1.5'
5+00	8°45'30"	
5+50	9°50'	-1.8'
6+00	10°46'30"	-4.2'
6+50	11°46'	-2.8'
7+00	12°49'	-3.1'
7+50	13°49'	-3.5'
8+00	14°49'30"	-2.7'
8+50	15°49'30"	-1.9'
9+00	16°51'	-2.0'
9+50	18°	-0.0'

NOTES:-

1. SLOPE VERTICAL DRAIN TOWARD LOW POINT AT STATION 5+44
2. CONTRACTOR TO VERIFY ALL DIMENSIONS IN FIELD AND MATCH EXISTING GEOMETRY WHERE APPROPRIATE
3. CONCRETE FROM DEMOLITION OF STRUCTURES MAY BE WASTED IN RESERVOIR AT DIRECTION ENGINEER
4. NEW SPILLWAY WILL BE 80 FEET WIDE WHERE OLD SPILLWAY IS 50 FEET WIDE

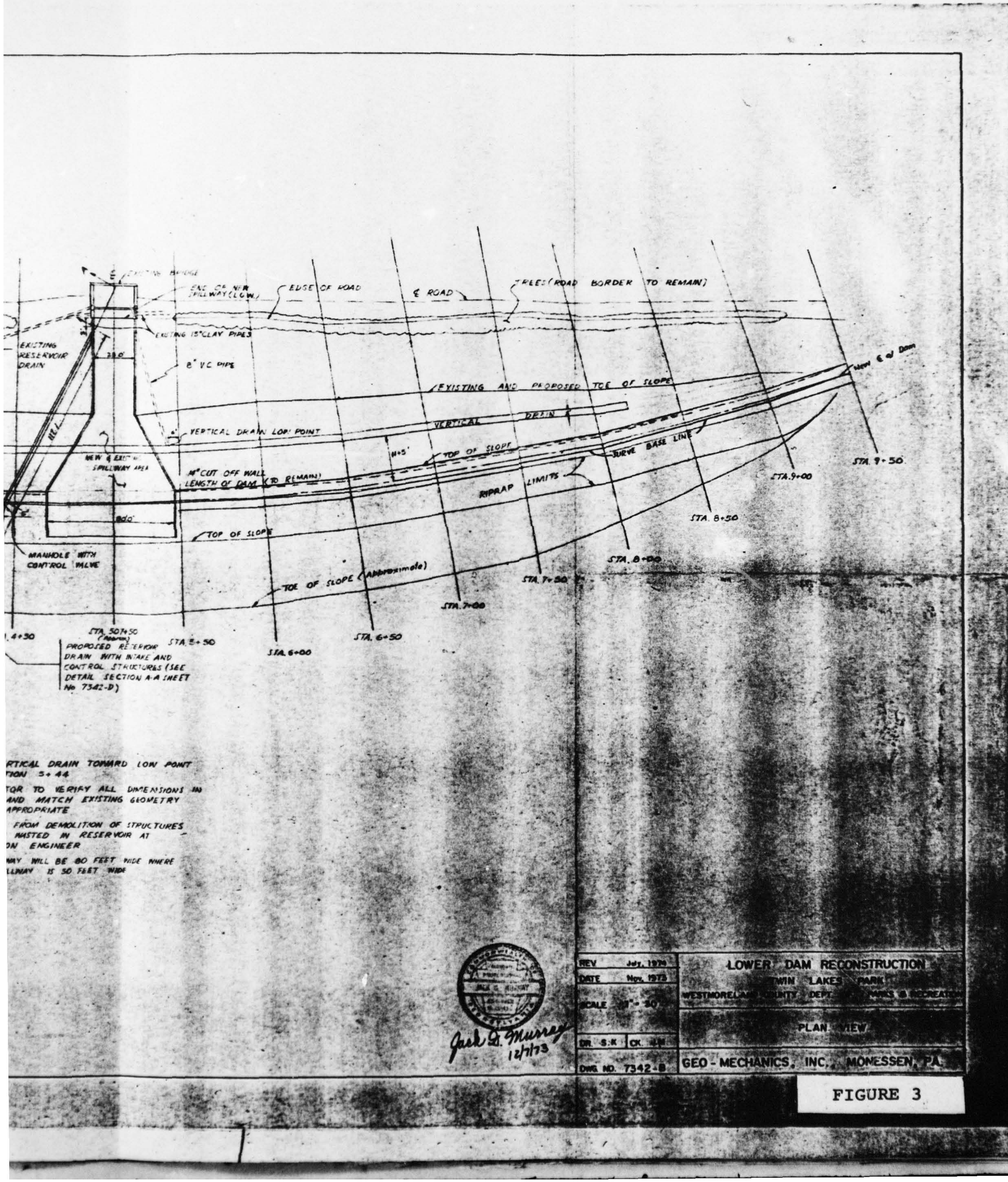
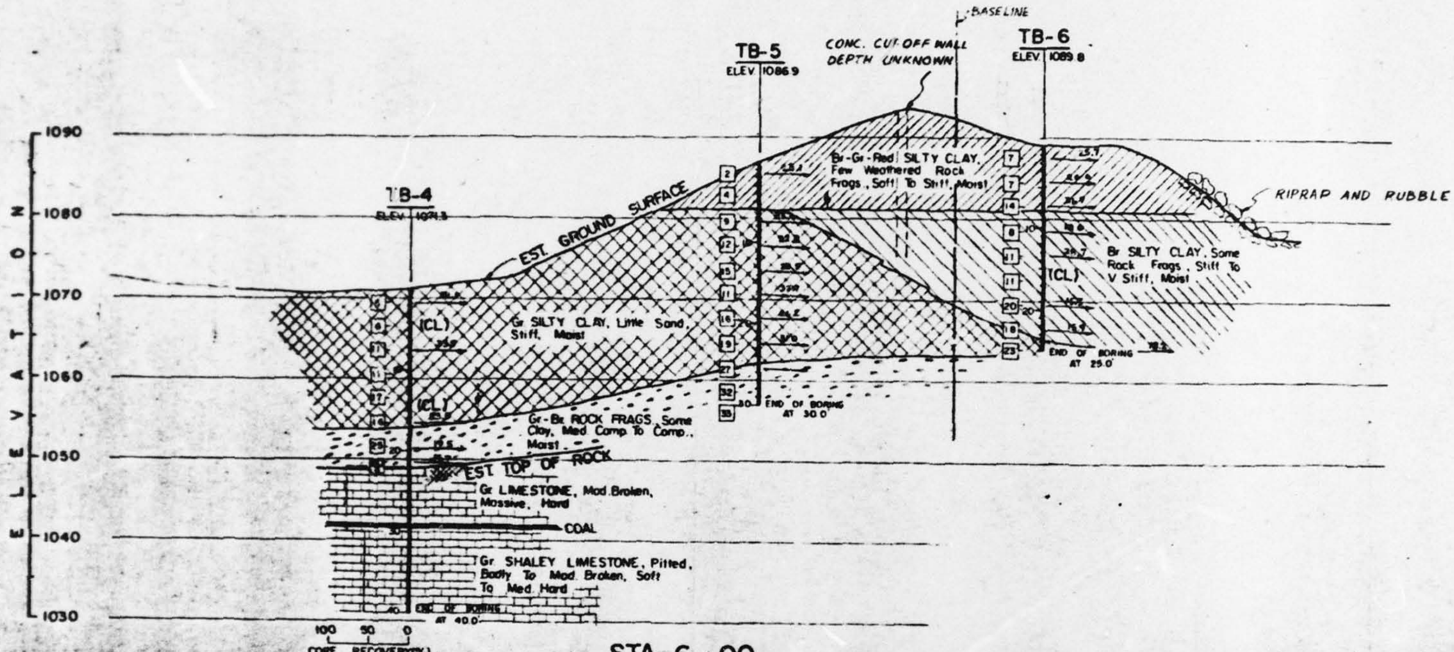
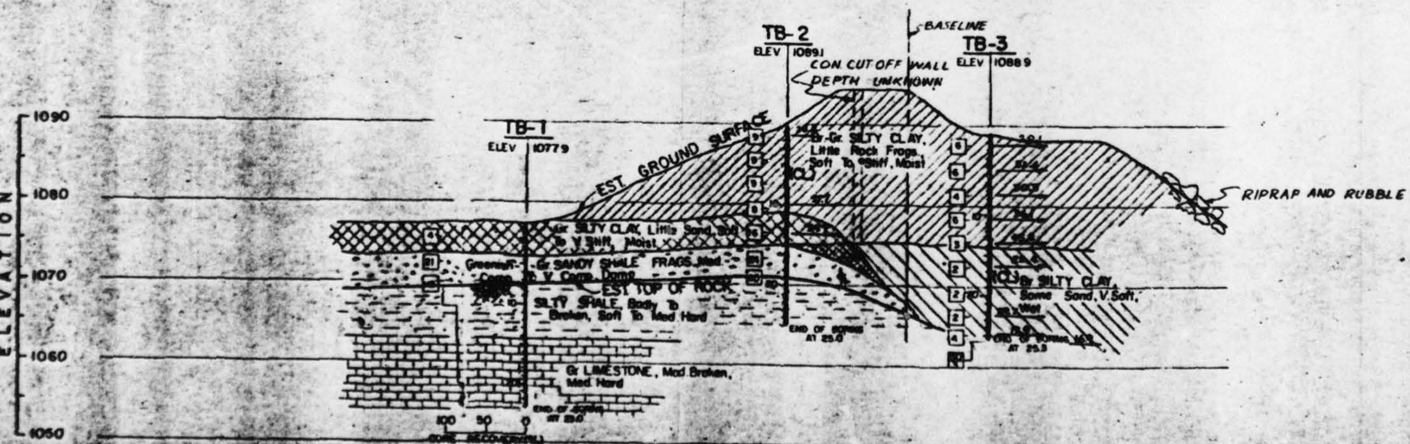

EXISTING BRIDGE
END OF NEW SPILLWAY (LOW)
EDGE OF ROAD
℄ ROAD
TREES (ROAD BORDER TO REMAIN)
EXISTING 15" CLAY PIPES
EXISTING RESERVOIR DRAIN
8" V.C. PIPE
EXISTING AND PROPOSED TOE OF SLOPE
VERTICAL DRAIN
VERTICAL DRAIN LOW POINT
TOP OF SLOPE
CURVE BASE LINE
New ℄ of Dam
NEW & EXISTING SPILLWAY AREA
CUT OFF WALL LENGTH OF DAM (TO REMAIN)
RIPRAP LIMITS
TOP OF SLOPE
MANHOLE WITH CONTROL VALVE
TOE OF SLOPE (Approximate)
STA. 9+50
STA. 9+00
STA. 8+50
STA. 8+00
STA. 7+50
STA. 7+00
STA. 6+50
STA. 6+00
STA. 5+50
STA. 5+00 (Approx)
4+50
PROPOSED RESERVOIR DRAIN WITH INTAKE AND CONTROL STRUCTURES (SEE DETAIL SECTION A-A SHEET No. 7342-D)
...RTICAL DRAIN TOWARD LOW POINT ...TION 5+44
...TOR TO VERIFY ALL DIMENSIONS IN ... AND MATCH EXISTING GEOMETRY ... APPROPRIATE
... FROM DEMOLITION OF STRUCTURES ... WASTED IN RESERVOIR AT ...ON ENGINEER
...WAY WILL BE 80 FEET WIDE WHERE ...LLWAY IS 50 FEET WIDE
Jack D. Murray 12/7/73
REV July, 1974
DATE Nov. 1973
SCALE 1" = 50'
DR. S.K. CK. J.M.
DWG. NO. 7342-B
LOWER DAM RECONSTRUCTION
TWIN LAKES PARK
WESTMORELAND COUNTY DEPT. OF PARKS & RECREATION
PLAN VIEW
GEO-MECHANICS, INC., MONESSEN, PA.

FIGURE 3

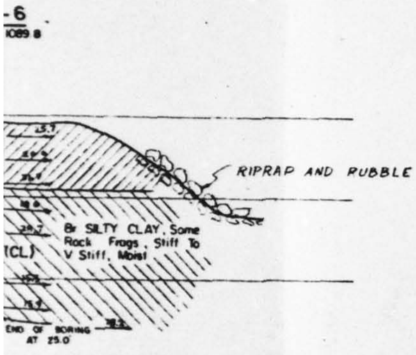


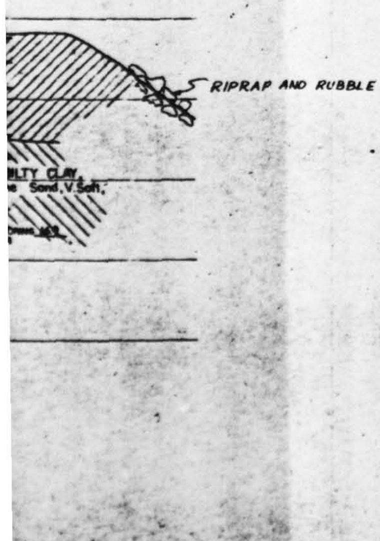
STA. 6 + 00



STA. 7 + 50

1.





LEGEND

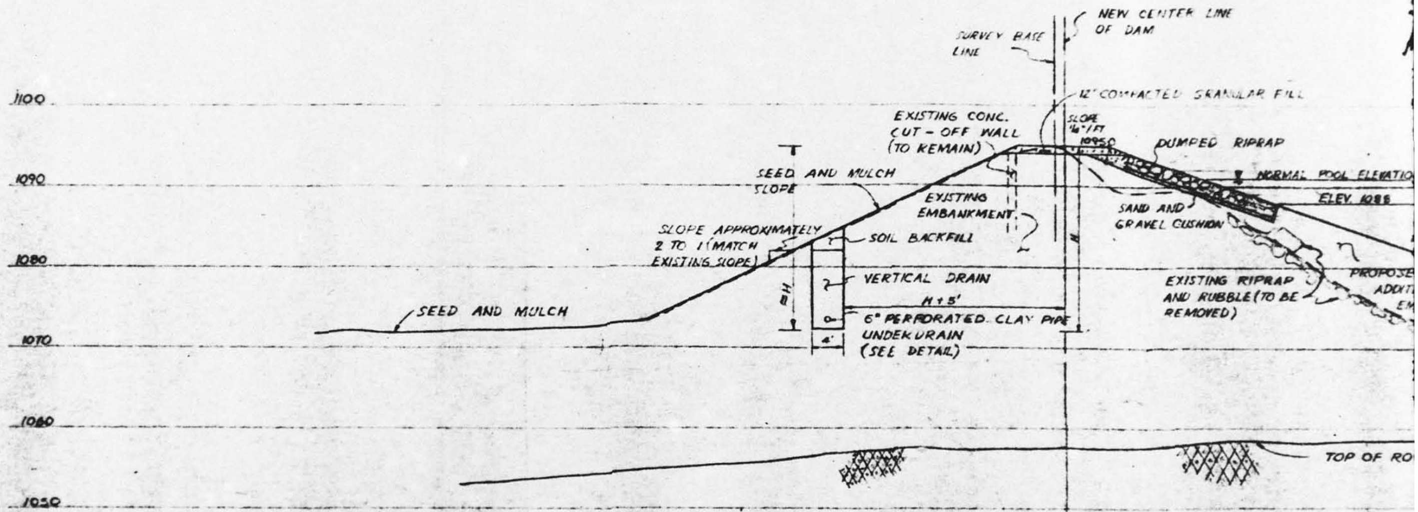
- SILTY CLAY
- SILTY CLAY W/LITTLE SAND
- SILTY CLAY W/ROCK FRAGS.
- ROCK FRAGS
- SILTY SHALE
- LIMESTONE
- COAL
- 7 PENETRATION RESISTANCE

The depth and thickness of the soil strata indicated on these Geologic Sections are generalized from and interpolated between the Test Borings. Information on actual subsurface condions exists only at the location of the Test Borings and it is possible that surface conditions between the Test Borings may vary from those shown.

REV.	LOWER DAM TWIN LAKES PARK WESTMORELAND COUNTY DEPT. OF PARKS & RECREATIO[N]
DATE Aug. 1973	
SCALE Ver. 1"= 10' Hor. 1"= 10'	GEOLOGIC CROSS SECTIONS AT STATIONS 6+00 AND 7+50
DR. S.K | CK. J.A	
DWG NO. 7342-4	GEO-MECHANICS, INC., MONESSEN, PA

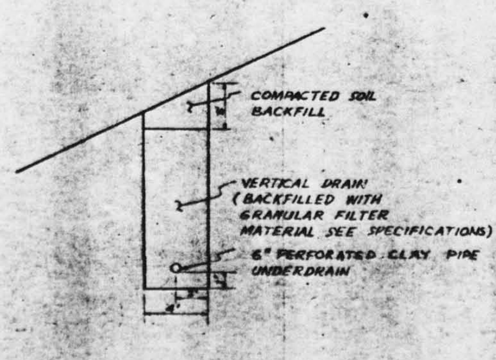
FIGURE 4

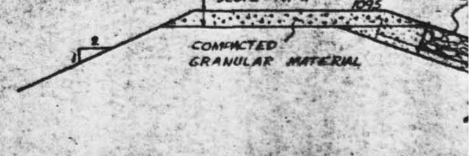
1.



TYPICAL SECTION

SCALE 1" = 10'



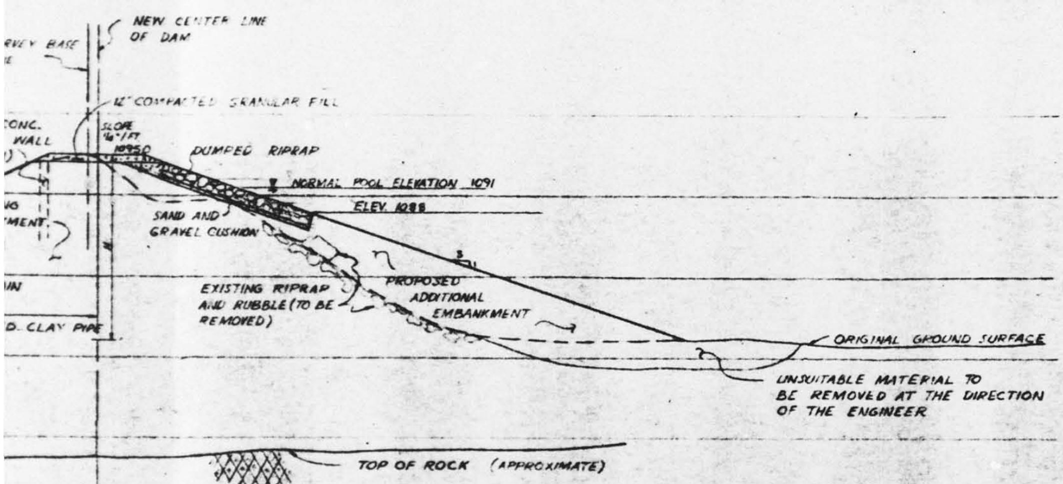


RIP RAP DETAIL

SCALE 1" = 5'

VERTICAL DRAIN DETAIL

SCALE 1" = 5'

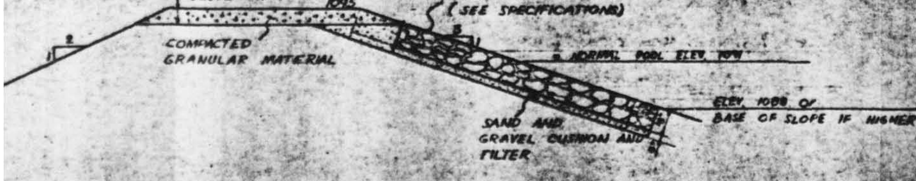


TYPICAL SECTION

SCALE 1" = 10'

NOTES:-

1. CONTRACTOR SHALL VERIFY ALL DIMENSIONS IN FIELD.
2. RUBBLE AND RIPRAP TO BE REMOVED FROM EXISTING SLOPE MAY BE WASTED IN THE RESERVOIR AREA AT THE DIRECTION OF THE ENGINEER
3. UNSUITABLE MATERIAL MAY BE WASTED IN THE RESERVOIR AREA AT THE DIRECTION OF THE ENGINEER
4. H = EMBANKMENT HEIGHT AT ANY SECTION
5. SLOPE VERTICAL DRAIN TO LOW POINT AT STATION 3+44
6. STOP VERTICAL DRAIN WHEN HEIGHT OF EMBANKMENT IS LESS THAN 10 FEET



RIP RAP DETAIL

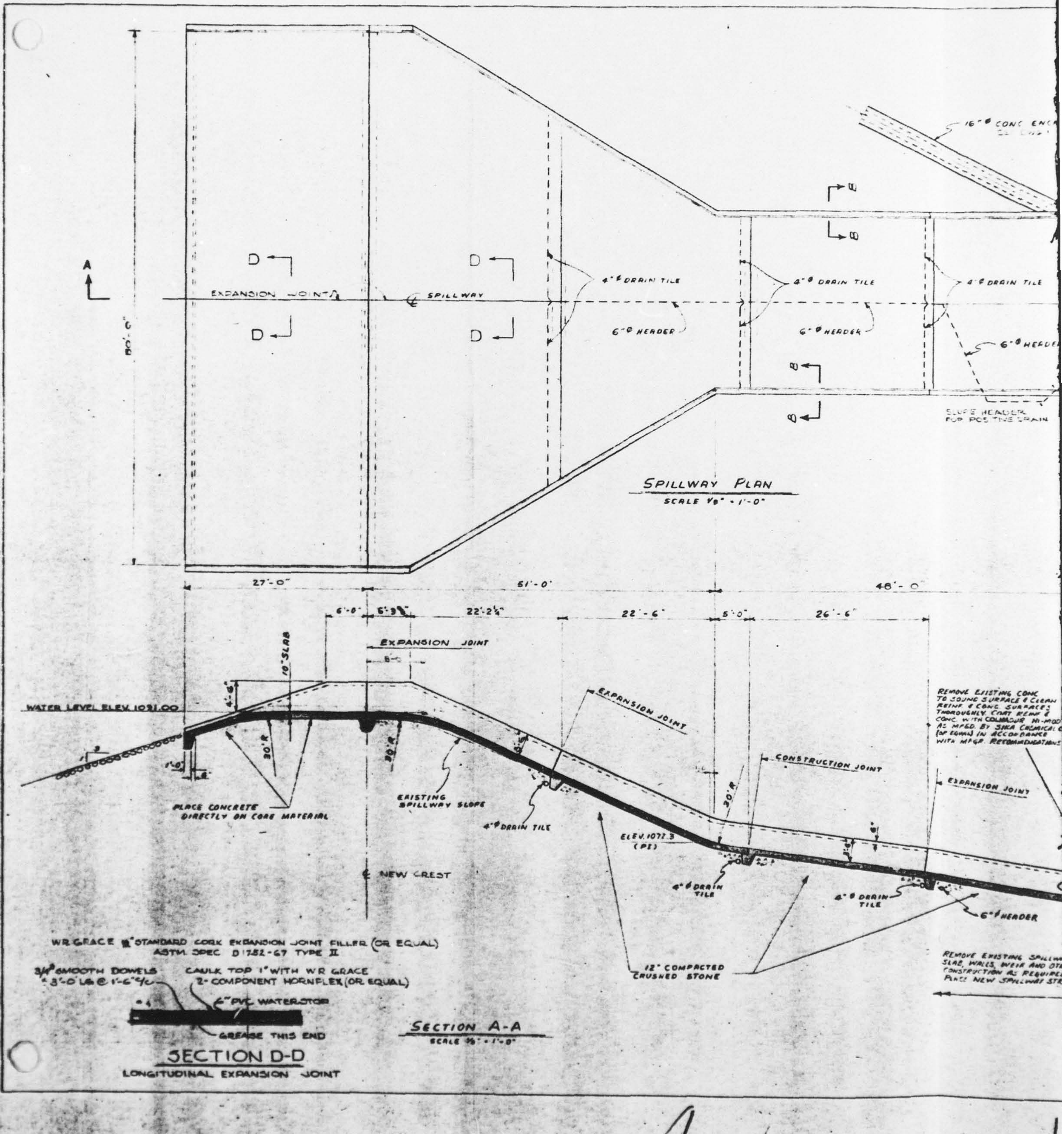
SCALE 1" = 5'

JACK G. MURRAY

Jack G. Murray
12/7/73

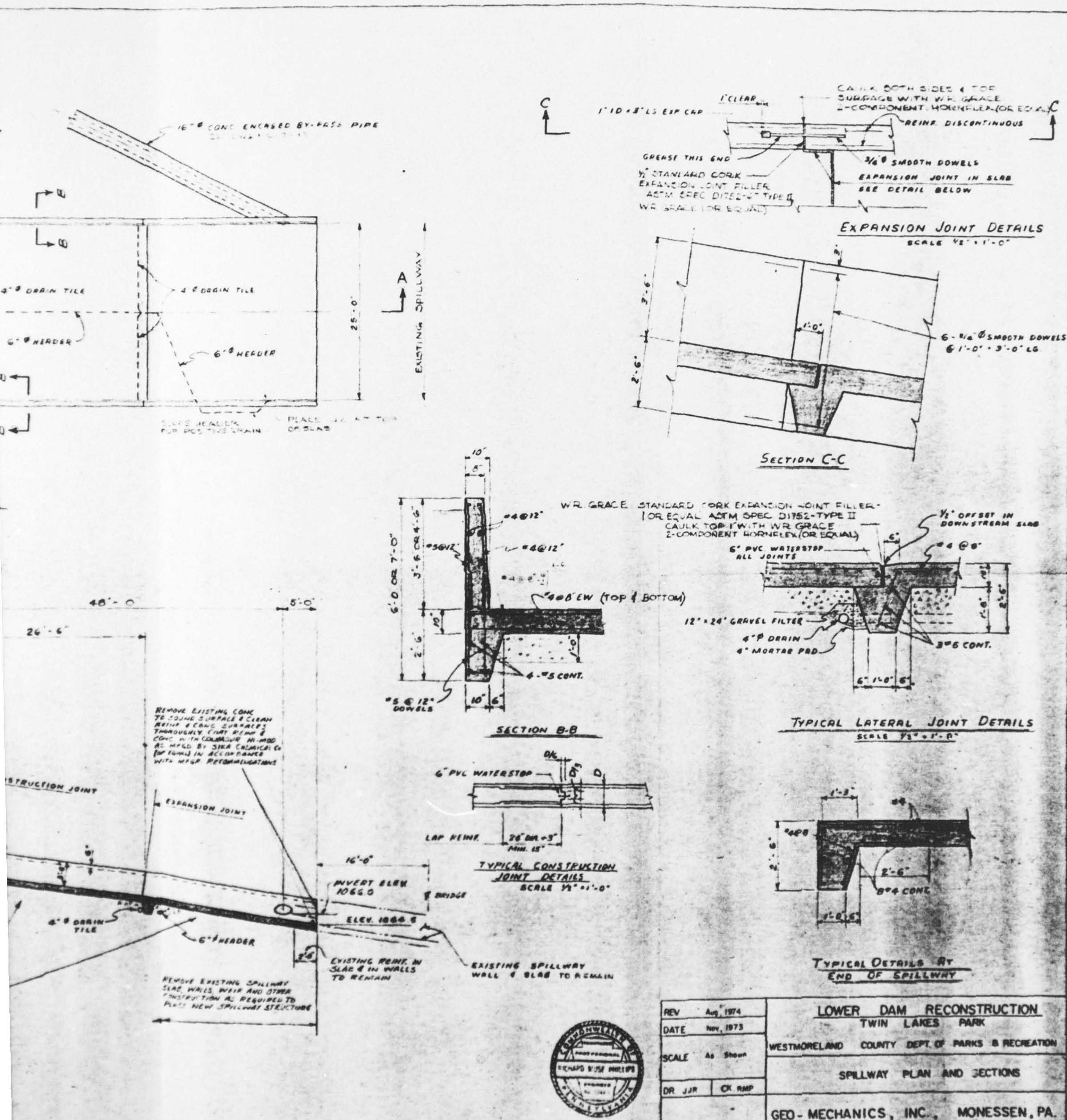
REV Aug. 74	LOWER DAM RECONSTRUCTION
DATE Nov. 73	TWIN LAKES PARK
SCALE As Shown	WESTMORELAND COUNTY DEPT OF PARKS & RECREATION
DR. S K / CK. J M	TYPICAL SECTION AND DETAILS
DWG. NO. 7342-C	GEO-MECHANICS, INC., MONESSEN, PA.

FIGURE 5



SPILLWAY PLAN
SCALE 1/8" = 1'-0"
EXPANSION JOINTS
SPILLWAY
4"Ø DRAIN TILE
6"Ø HEADER
SLOPE HEADER FOR POSITIVE DRAIN
16"Ø CONC ENC
80'-0"
27'-0"
51'-0"
48'-0"
6'-0"
5'-3 3/4"
22'-2 1/4"
22'-6"
5'-0"
26'-6"
EXPANSION JOINT
10" SLAB
WATER LEVEL ELEV. 1091.00
30'R
PLACE CONCRETE DIRECTLY ON CORE MATERIAL
EXISTING SPILLWAY SLOPE
4"Ø DRAIN TILE
NEW CREST
ELEV. 1072.3 (PI)
CONSTRUCTION JOINT
4"Ø DRAIN TILE
6"Ø HEADER
12" COMPACTED CRUSHED STONE
REMOVE EXISTING CONC TO SOUND SURFACE & CLEAN REINF & CONC SURFACES; THOROUGHLY COAT REINF & CONC WITH COLMASUR HI-MOD AS MFGD. BY SIKA CHEMICAL CO (OR EQUAL) IN ACCORDANCE WITH MFGR. RECOMMENDATIONS
REMOVE EXISTING SPILLWAY SLAB, WALLS, WEIR AND OTHER CONSTRUCTION AS REQUIRED. PLACE NEW SPILLWAY STR
SECTION A-A
SCALE 1/8" = 1'-0"
WR GRACE 1/2" STANDARD CORK EXPANSION JOINT FILLER (OR EQUAL) ASTM. SPEC. D1752-67 TYPE II
3/4"Ø SMOOTH DOWELS 3'-0" LG @ 1'-6" c/c
CAULK TOP 1" WITH WR GRACE 2-COMPONENT HORNFLEX (OR EQUAL)
6" PVC WATERSTOP
GREASE THIS END
SECTION D-D
LONGITUDINAL EXPANSION JOINT

1

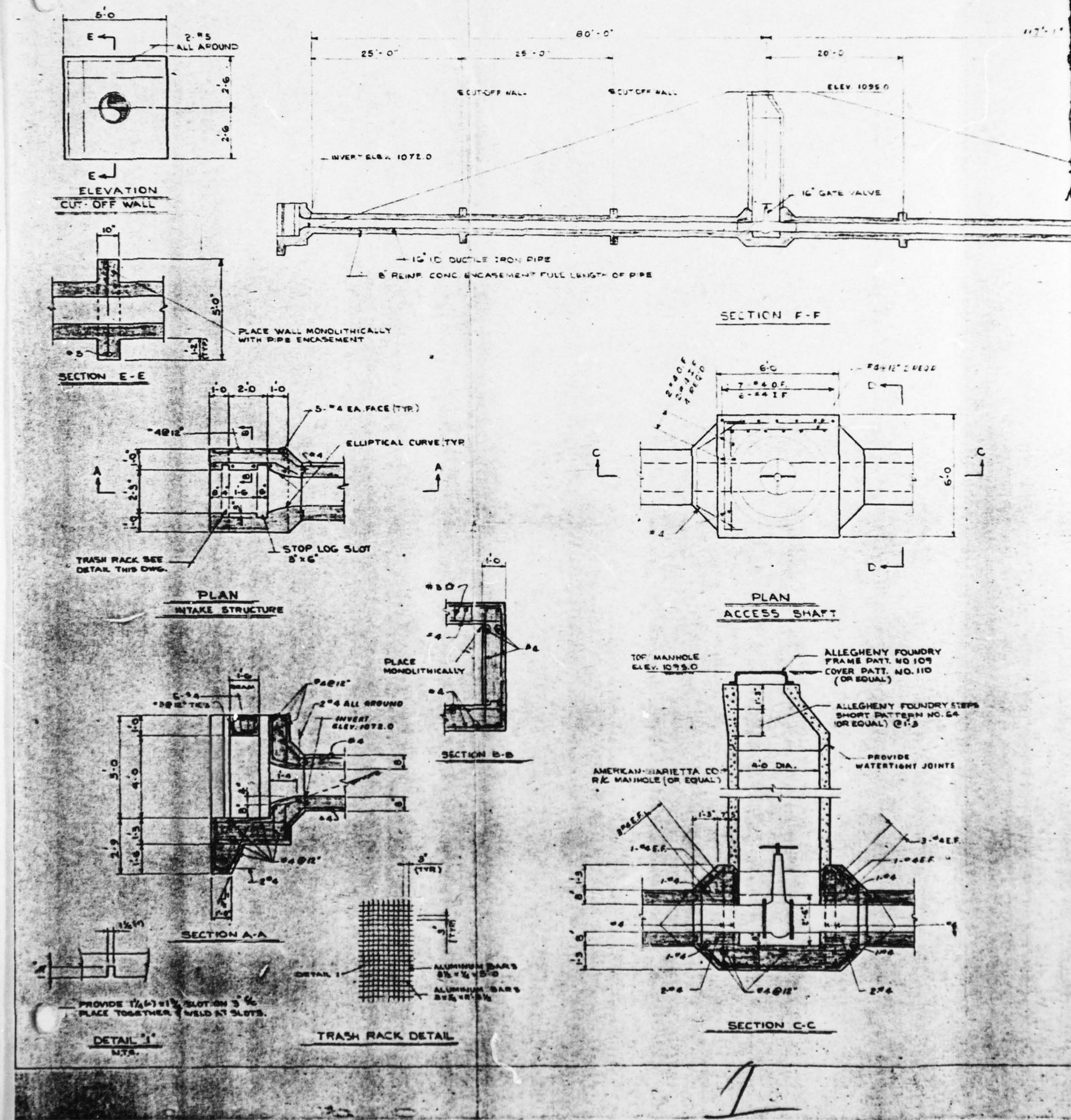


EXPANSION JOINT DETAILS
SCALE 1/2" = 1'-0"
SECTION C-C
6 - 3/4" Ø SMOOTH DOWELS @ 1'-0" - 3'-0" LG
REINF. DISCONTINUOUS
GREASE THIS END
3/4" Ø SMOOTH DOWELS
EXPANSION JOINT IN SLAB SEE DETAIL BELOW
W.R. GRACE (OR EQUAL)
16" Ø CONC. ENCASED BY-PASS PIPE
4" Ø DRAIN TILE
6" Ø HEADER
EXISTING SPILLWAY
48'-0"
26'-6"
5'-0"
25'-0"
SECTION B-B
#4@12"
#5 @ 12" DOWELS
4-#5 CONT.
#4@8" EW (TOP & BOTTOM)
W.R. GRACE STANDARD CORK EXPANSION JOINT FILLER (OR EQUAL ASTM SPEC D1752-TYPE II
CAULK TOP 1" WITH W.R. GRACE 2-COMPONENT HORNFLEX (OR EQUAL)
6" PVC WATERSTOP ALL JOINTS
1/2" OFFSET IN DOWNSTREAM SLAB
#4 @ 8"
12" x 24" GRAVEL FILTER
4" Ø DRAIN
4" MORTAR PAD
3-#6 CONT.
TYPICAL LATERAL JOINT DETAILS
SCALE 1/2" = 1'-0"
6" PVC WATERSTOP
LAP REINF.
TYPICAL CONSTRUCTION JOINT DETAILS
SCALE 1/2" = 1'-0"
EXPANSION JOINT
INVERT ELEV. 1066.0
ELEV. 1064.5
16'-0"
℄ BRIDGE
6" Ø HEADER
EXISTING REINF. IN SLAB & IN WALLS TO REMAIN
EXISTING SPILLWAY WALL & SLAB TO REMAIN
REMOVE EXISTING SPILLWAY SLAB, WALLS, WEIR AND OTHER CONSTRUCTION AS REQUIRED TO PLACE NEW SPILLWAY STRUCTURE
8-#4 CONT.
TYPICAL DETAILS AT END OF SPILLWAY
REV Aug. 1974
DATE Nov. 1973
SCALE As Shown
DR JJR CK RMP
DWG NO 7342-D
LOWER DAM RECONSTRUCTION
TWIN LAKES PARK
WESTMORELAND COUNTY DEPT. OF PARKS & RECREATION
SPILLWAY PLAN AND SECTIONS
GEO-MECHANICS, INC., MONESSEN, PA.

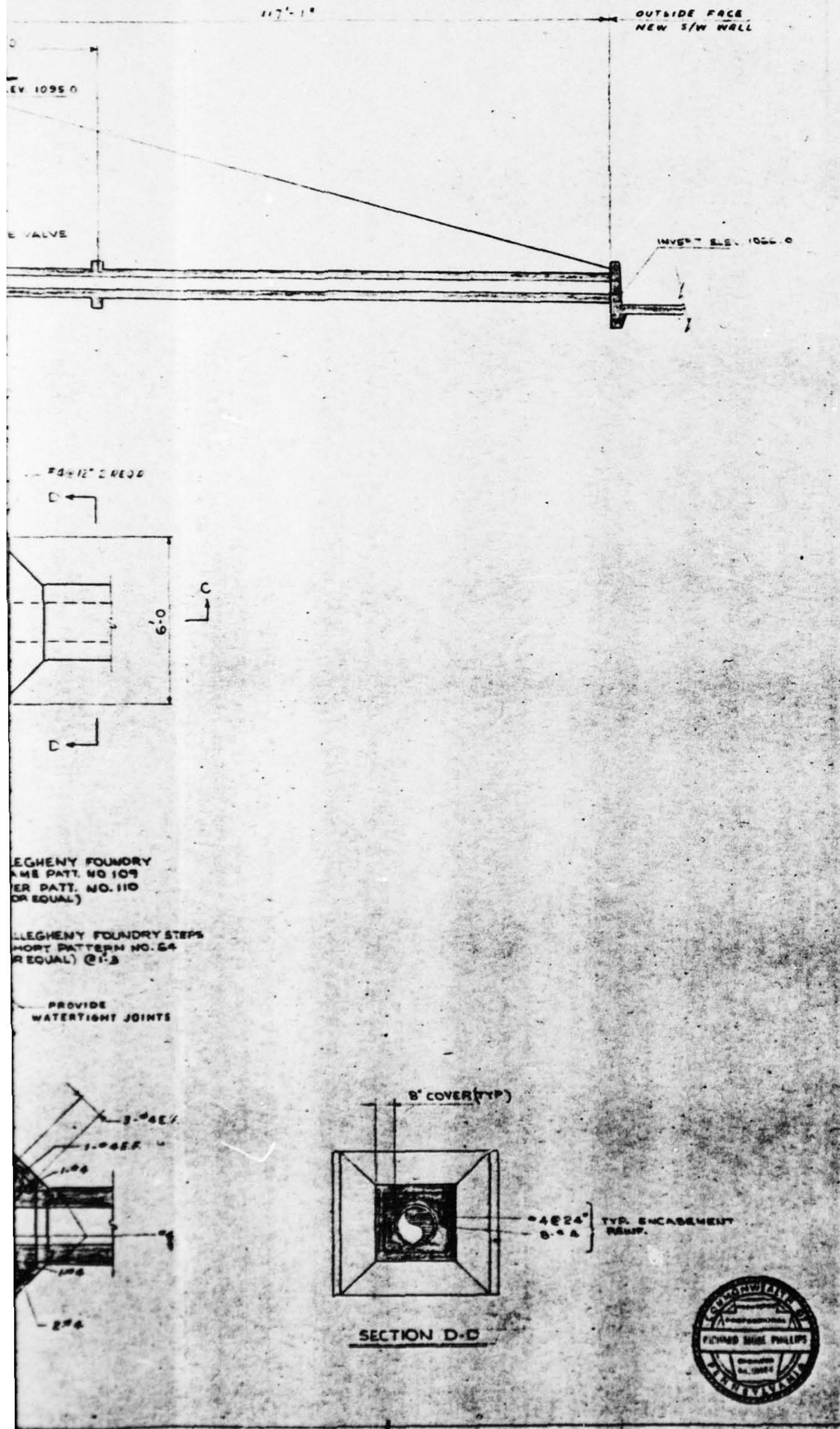
FIGURE 6

1

2



5'-0
2-#5 ALL AROUND
ELEVATION
CUT-OFF WALL
80'-0"
25'-0"
25'-0"
20'-0
℄ CUT-OFF WALL
℄ CUT-OFF WALL
ELEV. 1095.0
INVERT ELEV. 1072.0
16" GATE VALVE
16" I.D. DUCTILE IRON PIPE
8" REINF. CONC. ENCASEMENT FULL LENGTH OF PIPE
SECTION F-F
PLACE WALL MONOLITHICALLY WITH PIPE ENCASEMENT
SECTION E-E
5-#4 EA. FACE (TYP.)
ELLIPTICAL CURVE (TYP.)
STOP LOG SLOT 8"x6"
TRASH RACK SEE DETAIL THIS DWG.
PLAN
INTAKE STRUCTURE
PLAN
ACCESS SHAFT
PLACE MONOLITHICALLY
SECTION B-B
2-#4 ALL AROUND
INVERT ELEV. 1072.0
SECTION A-A
TOP MANHOLE ELEV. 1095.0
ALLEGHENY FOUNDRY FRAME PATT. NO 109
COVER PATT. NO. 110 (OR EQUAL)
ALLEGHENY FOUNDRY STEPS SHORT PATTERN NO. 64 (OR EQUAL) @1'-3
PROVIDE WATERTIGHT JOINTS
AMERICAN-MARIETTA CO. R/C MANHOLE (OR EQUAL)
4'-0 DIA.
SECTION C-C
ALUMINUM BARS
ALUMINUM BARS
DETAIL 1
TRASH RACK DETAIL
PROVIDE 1/4(+)x1 1/2 SLOT ON 3" C/C PLACE TOGETHER & WELD AT SLOTS.
DETAIL "1"
N.T.S.
1



GENERAL NOTES:

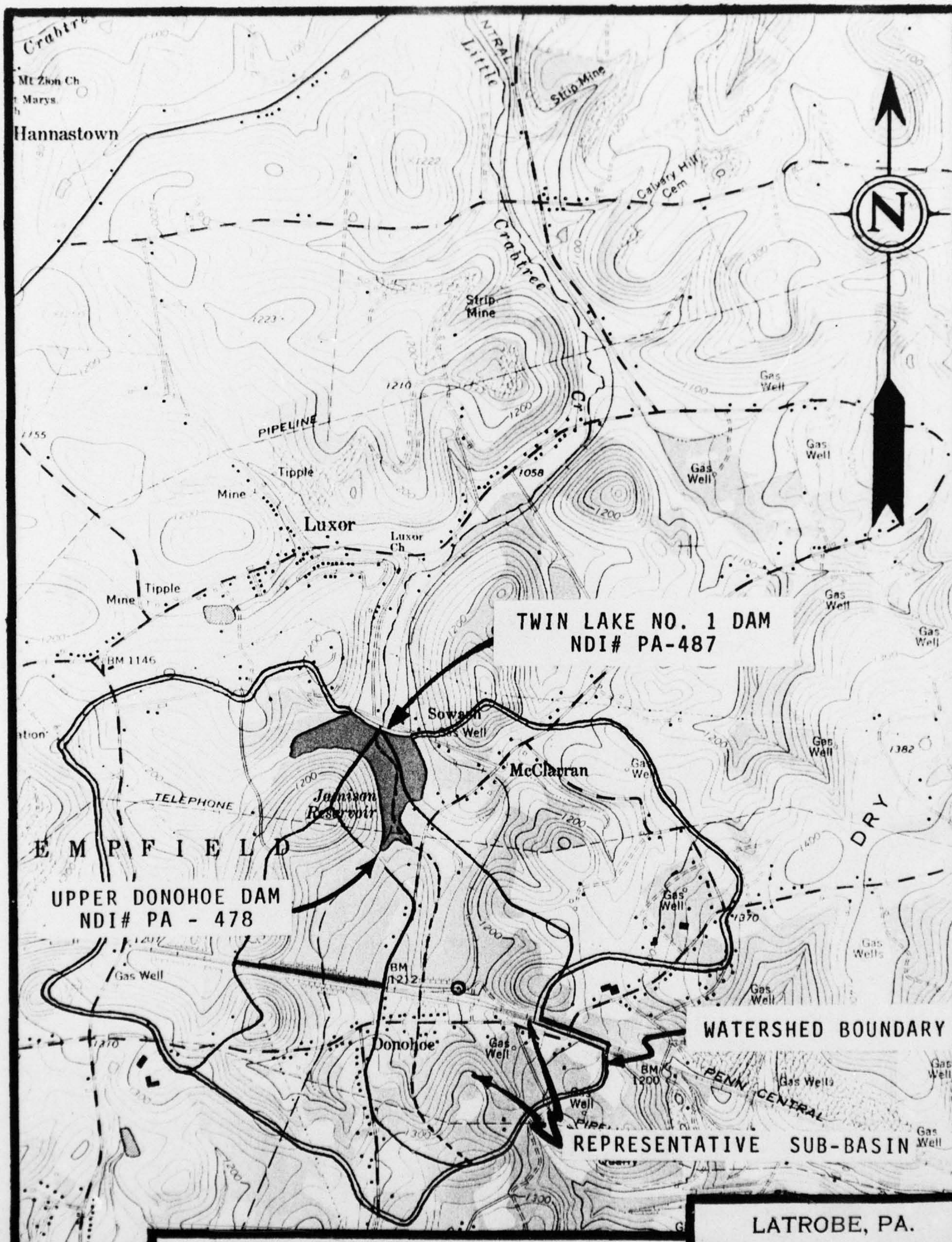
1 ALL CONCRETE SHALL HAVE A MINIMUM 4,000 PSI COMPRESSIVE STRENGTH IN 28 DAYS WITH 5 ± 1 PERCENT AIR ENTRAINMENT.
  CONCRETE PROPORTIONS:
    MIN. CEMENT FACTORS - 6 1/8 SACKS PER CUBIC YARD
    MAX. WATER CONTENT - 5 1/4 GALLONS PER SACK
    MAX. SLUMP - 3"
2 REINFORCING STEEL SHALL HAVE A MINIMUM YEILD STRENGTH OF 40,000 PSI AND CONFORM TO ASTM A615.
3 DETAILS, LAPS, BAR SUPPORTS, ETC. IN ACCORDANCE WITH ACI 315-65 UNLESS NOTED.
4 CONCRETE COVER ON REINFORCEMENT IN ACCORDANCE WITH ACI 318-71 UNLESS NOTED.
5 CONTRACTOR SHALL VERIFY ALL DIMENSIONS IN FIELD.
6 CHAMFER ALL EXPOSED CONCRETE EDGES 3/4" UNLESS NOTED
7 FURNISH ALUMINUM BARS OF ALLOY 6061-T6 AS MANUFACTURED BY ALUMINUM COMPANY OF AMERICA FOR USE IN THE TRASH RACK.

REV. Aug. 1974	LOWER DAM RECONSTRUCTION
DATE Nov. 1973	TWIN LAKES PARK
SCALE As Shown	WESTMORELAND COUNTY DEPT. OF PARKS & RECREATION
	OUTLET WORKS - PLANS, SECTIONS AND DETAILS
DR. / CK.	
DWG NO. 7342-E	GEO-MECHANICS, INC., MONESSEN, PA.

FIGURE 7

APPENDIX G

REGIONAL VICINITY AND WATERSHED BOUNDARY MAP



REGIONAL VICINITY MAP

LONGEST WATERCOURSE

CENTROID OF DRAINAGE AREA

LATROBE, PA.

N4015—W7922.5/7.5

1964

PHOTOREVISED 1973

AMS 5164 IV SW—SERIES V831